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Cleveland, Ohio

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DEMONSTRATION OF INTERIM TECHNIQUES FOR RECLAMATION OF POLLUTED BEACHWATER

by
James F. Weber
City of Cleveland
Cleveland, Ohio 44114

Grant No. 11020 EZW

Project Officer

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16. ABSTRACT

The shores of Lake Erie, bordering Cleveland, which had become contaminated because of combined sewer overflows, were made safe for swimming through the use of various interim techniques. These techniques were designed to keep beaches safe until long range goals such as an expanded sewage system can be implemented.

The swimming areas at White City and Edgewater beaches were disinfected with sodium hypochlorite and enclosed by fabric barriers. The barriers provide protection from contamination of the beaches by floating debris and retarded the diffusion of disinfectant into deep waters.

The other techniques which were attempted were: streams and combined sewer overflows were screened and treated with sodium hypochlorite; lateral sewers were flushed to reduce the "first slug" effect; physical clean-up of the beaches and water were made; and polymers were used in an attempt to increase sewer flow and thereby decrease sewer overflows. The phenomenon of "bacteriological aftergrowth" was investigated to find a method to optimize chlorination as a disinfectant.

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FOREWORD

The Environmental Protection Agency was created because of increasing public and government concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in problem solution and it involves defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment, and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources, for the preservation and treatment of public drinking water supplies, and to minimize the adverse economic, social, health, and aesthetic effects of pollution. This publication is one of the products of that research; a most vital communications link between the researcher and the user community.

In this study temporary measures were instituted to clean and maintain in a usable state public beaches that had become badly polluted. In addition, a number of techniques for improving the quality of wet weather discharges were investigated.

Francis T. Mayo
Director
Municipal Environmental Research Laboratory

ABSTRACT

The shores of Lake Erie, bordering Cleveland, which had become contaminated because of combined sewer overflows, were made safe for swimming through the use of various interim techniques. These techniques were designed to keep beaches safe until long range goals such as an expanded sewage system can be implemented.

The swimming areas at White City and Edgewater beaches were disinfected with sodium hypochlorite and enclosed by fabric barriers. The barriers provide protection from contamination of the beaches by floating debris and retarded the diffusion of disinfectant into deep waters.

The other techniques which were attempted were: streams and combined sewer overflows were screened and treated with sodium hypochlorite; lateral sewers were flushed to reduce the "first slug" effect; physical clean-up of the beaches and water were made; and polymers were used in an attempt to increase sewer flow and thereby decrease sewer overflows. The phenomenon of "bacteriological aftergrowth" was investigated to find a method to optimize chlorination as a disinfectant.

This report was submitted in fulfillment of Grant Number 11020 EZW, by the City of Cleveland, Ohio, Water Quality Program, under the partial sponsorship of the Environmental Protection Agency. Work was completed as of May, 1973.

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ACKNOWLEDGEMENTS

On behalf of those thousands of people who finally have the pleasure of safe swimming at Cleveland beaches, the author of the report would like to thank all those who were involved with the Demonstration of Interim Techniques for Reclamation of Polluted Beachwater.

This report was written by James F. Weber, Senior Chemist. Bacteriological examinations were performed by Charles Hina, Bacteriologist, and his staff, the chemical analysis were performed by Alex Balazs, Laboratory Director, and his staff of the Bureau of Wastewater Quality. Data for this report was compiled by Charles Hina and Paul E. Naton, Bacteriologists; Thomas C. Moore, Technical Specialist; William Shively, Engineer, Dennis Bechtel, Section Head, Bureau of Wastewater Collection; and Lamont Curtis, P.E., Havens and Emerson, Ltd., Consulting Engineers.

A special thanks goes to Frank Dumon, Commissioner of Parks, and Raymond Roth, Assistant Commissioner, Division of Water Pollution Control for originating the concept of a shoreline reclamation project and to Carl B. Stokes, former mayor of Cleveland and Ben Stefanski II, former Director of Public Utilities for their support in getting the project underway.

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SECTION I

CONCLUSIONS

The installation of fabric barrier enclosures at bathing areas and disinfection of these enclosed areas by chlorination produced acceptable water quality for recreational purposes.

The fabric barriers were effective in preventing the infiltration of debris into the bathing areas and onto the beaches. The more continuous construction of the Edgewater barrier was more effective at this than the White City Beach barrier.

Less maintenance was required on the Edgewater Beach barrier which, being less buoyant than the White City barrier, allowed it to submerge during periods of rough water and prevented damage to the barrier.

Automatic chlorination used in conjunction with a fabric curtain barrier was shown to be a highly effective method for improving water quality for swimming and recreational purposes.

Without chlorination and enclosure, the water quality at both beaches under study would have been unsafe for human recreational purposes, according to Ohio State water quality standards.

By enclosing the bathing areas and then chlorinating inside the enclosure, the fecal coliform density was controlled below the maximum allowable level (200 per 100 ml) during periods of rainfall and overflow as well as dry weather.

High bacteria counts occurred only when the level of residual chlorine inside the enclosure was zero. In such cases, the fecal coliform density inside the enclosure was, at times, greater than the density outside the enclosure. This is attributed to the bacterial after-growth phenomenon.

Levels of residual chlorine should be above 0.2 ppm for maximum effectiveness, but below 1.0 ppm so as to avoid possible injury to bathers.

Barriers and chlorination are only a temporary measure to improve water quality until the real problem of storm overflows and inadequate sewage treatment can be corrected.

As a result of hypochlorination of streams and overflows, there was a definite reduction in pollution indicator bacteria, fecal coliforms, fecal streptococci, and total coliforms, but little or no reduction of these bacteria in the streams at their point of entry into Lake Erie.

Although a large percentage of fecal coliform bacteria was killed during periods of storm overflow due to chlorination, the reduction of pollution indicator bacteria was far from adequate to meet the State of Ohio stream water criteria for recreational purposes.

Data from stream sampling stations during periods of hypochlorination indicates that hypochlorination of streams results in a significant reduction of pollution indicator bacteria; however, as soon as the chlorine residual dissipates, a bacterial aftergrowth occurs producing bacterial numbers as great and sometimes greater than the initial bacteria levels.

Factors found to significantly affect bacterial aftergrowth are: (1) the extent of dilution of the chlorinated water; (2) time available for aftergrowth between chlorination and dilution; and (3) levels of residual chlorine.

While maintaining a 6 mg/l chlorine residual throughout a laboratory study, no significant decrease in aftergrowth was noted by increasing the chlorination detention time from 15 minutes to 72 hours.

Proteins, as analyzed by the Lowry Method for protein determination, were greatly increased in stream water samples upon the addition of sodium hypochlorite. It is hypothesized that chlorine cleaves larger protein molecules into smaller proteins, peptides, and amino acids which yield more reactive sites to react with the Lowry color development reagent. All the reactive nitrogenous organic compounds were calculated as protein. Since both laboratory and field studies show bacterial populations were greater after chlorination than before, it is further hypothesized that the smaller nitrogenous compounds were more easily utilized by the bacteria for growth and reproduction which could be significant in the rate and magnitude of bacterial aftergrowth.

Other than of proteinaceous material, there were no appreciable chlorination induced chemical-physical changes in the water samples studied that could be demonstrably related to bacterial aftergrowth.

Particle size of suspended solids had little affect on bacterial aftergrowth.

In municipalities with large numbers of overflows, sewer flushing combined with street washing could prove effective in reducing pollution loads to the area's receiving waters. Since a large amount of suspended solids and Chemical Oxygen Demand (COD) containing material can collect in residential, lateral, or combined sewers reducing these concentrations in the sewer during dry weather reduces the load which would otherwise reach the surface waters during wet weather.

Large outlays of capital are not necessary to perform the sewer flushing procedures. Unskilled laborers could perform this task with a minimum of equipment.

The sewer flushing procedure, using a tank truck to first wash the streets, was shown to be the best method for sewer flushing in this study because it realistically combines the removal of street pollutants with sewer flushing.

Methods of direct flushing of sewers with firehoses using water obtained from a fire hydrant also proved effective in cleaning sewers.

Although the addition of polymers have been shown to increase the flow velocities in pipes flowing full, according to the results of this study, flow velocity in sewers was not increased. It was concluded that the failure of polymer addition to increase flow velocity could have been due to lack of sensitivity of the flow measurement system.

A screening device approximately one mile from the mouth of Nine Mile Creek has proven to be an effective means in reducing a quantity of floating trash and debris which would have been discharged into Lake Erie. It is not possible to exactly determine to what extent Cleveland's beaches have benefitted from this operation. However, as a result of the screening operation, approximately ten tons of trash were removed from the Creek during the spring to autumn operating season.

A dual program of debris removal on the Cuyahoga River from land as well as marine access is more effective and cost less than operations which are limited to waterside removal only.

It is of interest to note that the effect of dilution on reducing peak aftergrowth was greater than would have been expected on the basis of dilution ratios alone. For example, the average peak aftergrowth of total bacteria in 75% Dugway Brook water was only 34% of the peak aftergrowth in 100% Dugway Brook water. This affect was apparent for all dilutions and for each group of bacteria, except for fecal streptococci.

SECTION II

RECOMMENDATIONS

Since the Demonstration of Interim Techniques for Reclamation of Polluted Beachwater program was successful in providing safe recreational beaches, it is recommended that the equipment used in this project be turned over to the Division of Recreation for their use in providing safe beaches in the future.

A new flotation system for the rigid White City Beach barrier should be designed to replace the one now in use. The new flotation system should be less buoyant and more continuous in construction to reduce the repair cost and increase beach protection from debris.

Efforts should be made to reduce yearly installation and seasonal maintenance costs. The presently designed barriers are difficult to handle and are not conducive to frequent manipulation.

It is recommended that until the cause of the unsanitary lakefront is eliminated, the hypochlorination project for all beaches be continued, since it was proven that the overall project was successful and that the methods for assuring safe swimming conditions are feasible.

Breaks in the Easterly Overflow conduit caused high bacteria densities along the conduit inside the White City Beach enclosure. The breaks in this conduit should be repaired.

Edgewater Overflow should be eliminated from the Edgewater Beach area since it was shown that this overflow contributes the greatest pollution load to this bathing area.

A supply of solid calcium hypochlorite should be available for use during emergencies such as mechanical breakdown of electrical pumps or interruptions of supply of the sodium hypochlorite solution.

It is recommended that all sewer overflows that empty into the beach areas along the lake be intercepted and re-routed to a sewage treatment plant.

It is recommended that hypochlorination of streams be discontinued since this method of mass disinfection was shown to be ineffective.

Included in a program for in-stream chlorination should be information concerning: (1) the degree of dilution of chlorinated waters; (2) stream-flow time from the chlorinator to the point of outfall; (3) quality of the diluting water; and (4) the desired level of residual chlorine.

If the degree of dilution of chlorinated water is not greater than 1/5, or the dilution will not occur within 24 hours of chlorination, it is recommended that in-stream chlorination should not be attempted.

Further experimentation should be conducted to determine the mechanics of bacterial aftergrowth and in particular the rate of controlling effects of hydrolysis and cleavage of proteins by chlorination.

A study to determine how quickly a "flushed" sewer reacquires large amounts of settled solids should be undertaken to find out how long a sewer can stay "cleaned".

If a sewer does not accumulate a high concentration of settled solids for several days, it is recommended that a program coordinating street flushing and sewer flushing in dry weather should be adopted as a means of reducing pollution caused by combined sewer overflow during precipitation periods.

Several streets should be washed and flushed simultaneously, both to eliminate resettling in larger sewers and to produce a sufficient flow to carry pollutants to the sewage treatment plant.

A thorough theoretical analysis of polymer-water flow phenomena in open-channel flow should be completed before more field trials are attempted.

The Nine Mile Creek screening device, which was a manually hoisted basket, was effective in removing trash from the creek during dry weather flow. As flows increased beyond the dry weather rate, more of the total stream flow bypassed the screen without receiving in-stream removal. During peak flows when the creek had swollen to a width of fifty feet or more, only some ten percent of the stream surface area was being screened. Hydrological data should be analyzed before designing any future screening devices.

The downstream side of the inverted syphon structure on Nine Mile Creek is an excellent site for a basket screen since stream flow must pass under a large interceptor sewer at this point. The syphon prevents the more buoyant forms of debris from flowing further downstream by catching it in the structure's influent stilling pool. The debris floats at the surface of the pool until higher rates of flow raise the water level. The floating trash is then spilled into weir-like traps on either side of the syphon pool. As a result, the types of debris which are allowed to pass downstream from the syphon are limited to leaves, rags, paper and similar material which can be easily screened. The device would be similar to the existing screen in construction and operation, except that it would be wide enough to allow for all flow conditions.

Another approach for screening would be to construct a stationary screen with a movable apparatus which would automatically clean the screen at predetermined intervals or at frequencies corresponding to the rate of flow. The trash could be deposited in solid waste disposal bins for easy transport. Initial cost of a more automated process obviously would be greater, but the savings realized in reduced operating costs should favorably offset the added cost of automation.

Design and operation of trash removing equipment will vary for every system and overflow because each has a peculiar set of characteristics. In designing a screen, every aspect of that stream or overflow should therefore be considered. Existing structures should be investigated for possible utilization to avoid unnecessary expenditures. Cost versus performance comparisons, in which initial equipment costs, maintenance, and disposal costs are compared to tonnage removal and benefits, should be considered closely.

A continuing debris removal program should be conducted concurrent with recreational waters improvement programs.

Control at the beaches alone will not produce debris free water. The program should be extended from the recreational season to cover the entire navigational season, that is, March through December.

A permanent debris and oil removal facility should be constructed on the Cuyahoga River upstream of the navigational channel. This area is well suited for such an installation. The head waters of the dredged navigational channel is where debris is most frequently found in abundance.

Permanent oil removal capability should be available for routine clean up as well as emergency spill containment and recovery.

Although partially effective, the simultaneous removal of oil with debris or with absorbants should not be considered as an adequate oil removal program.

Double handling of debris should be eliminated by utilizing containerized units which could be lifted directly from the boat to a land-side disposal vehicle.

The cleaning of the beaches should not be limited to the bathing season; debris should be removed at frequencies corresponding to the rate of accumulation and as weather permits.

SECTION III

INTRODUCTION

There are two beaches along the shore of Lake Erie in the City of Cleveland. White City Beach, located on the City's east side, has a shoreline of 700 feet. Edgewater Beach, on the City's west side, has a shoreline of over 1000 feet. Both White City and Edgewater have natural soft, sandy beaches. The water at the beaches increases gradually in depth to about twelve feet. Often two or three foot breaking waves caused by north winds would roll in toward shore, providing excitement as bathers would "challenge" the surf. The aesthetically pleasing quality of the beaches and their close proximity to highly populated areas lead to their frequent and continual use by the people of the Cleveland area.

By the 1960's, White City and Edgewater Beaches had become so polluted they could no longer be safely used. The problems of these polluted beaches are commonly known: Overflows from combined sewers loaded the beaches and beachwater with organic and inorganic material and nutrients that would support a high infectuous bacterial population; dead and decaying fish, flotsum and algae became incorporated into an odorous mass that added to the festering beach and water. And, ultimately, driftwood, dead rats, and rotting seaweed covered these once highly frequented beaches.

The nearest safe beaches are 15 to 20 miles away from Cleveland. These beaches are inaccessible to large numbers of innercity youths who were forced to use already overcrowded swimming pools.

In 1966, Commissioner of Parks Frank Dumon, approached Raymond Roth, Chief of the Bureau of Industrial Wastes and Assistant Commissioner of the Division of Water Pollution Control, and asked if something could be done about this intolerable situation with the beaches. Raymond Roth, through the cooperation of the Honorable Carl B. Stokes, Mayor of the City of Cleveland, the Department of Public Utilities, and Havens and Emerson, Ltd., Consulting Engineers, formulated the Demonstration of Interim Techniques For Reclamation of Polluted Beachwater. Funding by the Environmental Protection Agency (at that time, the Federal Water Pollution Control Agency) was requested and granted.

To clean beaches in such poor condition as White City and Edgewater required innovative techniques to strike out at both the causes and symptoms of this massive water pollution problem.

Physical barriers had to be devised to separate the lake water from the beachwater. The barriers would provide protection from flotsom from combined sewer overflows and retard the disinfectant from diffusing into deep water.

Floating debris was removed by hand or seined from the water with minnow nets. A bulldozer and dumptruck were used to remove logs and tree trunks washed onto the beach, to allow a mechanical beach sanitizer to keep the beaches free of debris.

Sodium hypochlorite was chosen for the disinfection system. This disinfectant, diluted with fresh water, provided a positive displacement of the water enclosed by the barriers to reduce contaminated lake water from infiltrating into the enclosure.

To reduce the bacterial levels along the City's lakefront, streams and storm overflows with outfalls near the swimming areas were chlorinated. To increase the effectiveness of stream and overflow chlorination, the phenomenon of bacterial aftergrowth was investigated.

Three methods of reducing lakefront contamination were attempted during this project. They were: (1) the reduction of combined sewer overflow by the addition of "flow increasing" polyelectrolytes into an interceptor sewer; (2) flushing lateral sewers in residential areas during dry weather to reduce the "first slug effect" caused by the purging and storm overflow of settled solids during a rainstorm; and (3) cleaning up of debris in rivers and streams before it washed into the Lake.

This Demonstration of Interim Techniques for Reclamation of Polluted Beachwater Project covered a span of three years from 1968 through the summer of 1970. A chronology of significant events is as follows:

1967	The Department of Public Utilities and Havens and Emerson, Ltd., Consulting Engineers, formulated the project and petitioned the F.W.P.C.A. for financial assistance.
April, 1968	F.W.P.C.A. accepted the program and granted financial assistance.
July, 1968	White City Beach barrier was installed. Chlorination made bathing safe.

July, 1968	Construction of the hypochlorination facilities for streams and overflows was initiated.
September, 1968	Project was ended for the season.
May, 1969	Screening device on Nine Mile Creek was in use.
June, 1969	The barrier at Edgewater was installed and chlorination of the enclosed waters made bathing safe. Because of a tugmens' strike White City barrier was not installed. Hypochlorination of streams and overflows began.
July, 1969	Polymer study initiated.
September, 1969	The beaches are closed for the season. The polymer additions to increase sewage velocity to decrease storm overflows was begun.
June, 1970	The curtains were installed at White City and Edgewater. Aftergrowth study began. Hypochlorination of streams continued.
August, 1970	Sewer Flushing project started.
September, 1970	The Demonstration Project For The Reclamation Of Beaches ended.

SECTION IV

POLLUTION BARRIERS AND BEACHWATER QUALITY

INTRODUCTION

The major objective of this Demonstration Project of Interim Techniques For Reclamation of Polluted Beachwater was to make two beaches in the City of Cleveland fit for swimming. In the first step in attaining this goal, (1) barriers were constructed at White City and Edgewater Beaches to physically separate the beachwater from the rest of the lakefront; (2) the water enclosed by the barriers was disinfected; and (3) the sandy beaches were reclaimed by the use of mechanical debris collectors.

THE BARRIERS

The barriers constructed to enclose the beachwater were nylon curtains suspended in the water by buoys. In order to evaluate the effectiveness of the curtain design, two different types of plastic curtains were fabricated.

At White City, a curtain with a rigid construction was installed, and at Edgewater, a curtain with more resilience and less buoyancy was installed.

THE BARRIER AT WHITE CITY BEACH

White City Beach, on Cleveland's east side, was chosen as the original site. Its bathing area is partially protected by stone breakwalls which limit wave action. At the beach (Figure 1), interlocking sheet steel pilings were installed to form a wall perpendicular to the beach and parallel to an existing solid pier and storm overflow conduit. The piling extends 490 feet into the lake and forms the western boundary of the enclosed area. The curtain was transported to the beach area by truck, was placed into the lake by a crane, and towed into position for installation by boat. When installed, the curtain spanned the 700 foot distance between the sheet piling and the pier at a depth of 12 feet to completely enclose the eight acre swimming area.

The barrier, designed by the Havens and Emerson, Ltd.. Consulting Engineers, and constructed by Samsel Rope and Marine Supply, consisted of nylon fabric panels twenty-three

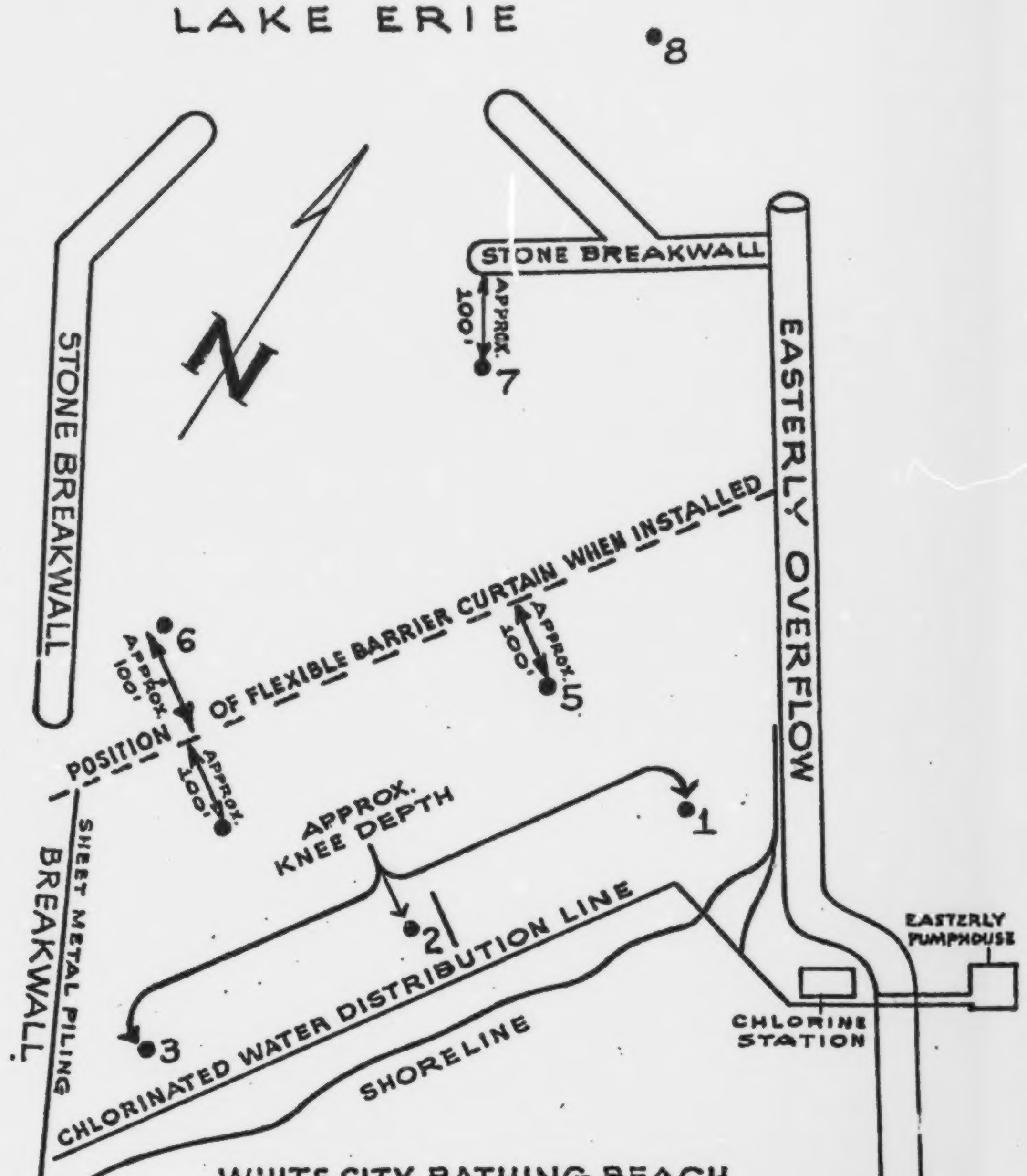

LAKE ERIE
8
STONE BREAKWALL
N
STONE BREAKWALL
APPROX. 100'
7
EASTERLY OVERFLOW
POSITION OF FLEXIBLE BARRIER CURTAIN WHEN INSTALLED
6
APPROX. 100'
APPROX. 100'
5
APPROX. 100'
APPROX. KNEE DEPTH
1
2
3
SHEET METAL PILING BREAKWALL
CHLORINATED WATER DISTRIBUTION LINE
SHORELINE
EASTERLY PUMPHOUSE
CHLORINE STATION
WHITE CITY BATHING BEACH
● SAMPLING LOCATIONS

Figure 1.

White City Beach swimming area

feet long, laced together with nylon rope. Each of the thirty-one panels was suspended at the top by two corrugated, galvanized tubes which were filled with styrofoam for buoyancy. These galvanized tube pontoons were twenty inches in diameter and ten feet long. All were fastened together by a single length of wire cable seven hundred feet in length.

The enclosed area was dredged until the depth of water increased gradually to a depth of 12 feet. At the 12 foot level, "H" pilings were pile driven vertically to provide anchorage for the fabric barrier. Adjustable brackets were mounted on the "H" piling to provide uniform taut mooring, as shown on Figure 2.

The barrier was effective in a definite reduction of the floating debris within the swimming area and on the beach. Since less debris entered the bathing area, the beaches were more easily cleaned, more attractive, and more frequented by the public. The barrier also was successful in preventing the mixing of lake water with the disinfected beachwater allowing for longer disinfectant residuals.

There were several problems associated with the barrier at White City Beach, however. During the first year of operation, the barrier experienced considerable damage as a result of wave action upon its rigid construction. The pitching action of the pontoons ripped or wore out anchor straps and lacings at other stress points, and in 1969, the barrier was not installed because of a tugmens' strike.

Prior to the 1970 bathing season, the barrier at White City was modified to lessen its rigidity and increase its strength. Springs and rubber automobile tires replaced the metal connection swivels between pontoons. Existing anchor straps were reinforced and snap ring connectors were replaced by heavier shackles. As a result of these changes, very few anchor straps were broken during the 1970 season. The anchor cable was broken twice, anchor plates were pulled from their "H" beam mooring, and some panels were ripped in two, but most of the anchor straps and pontoon connections remained intact. The cost of the barrier installation and repair for 1970 was $17,500.

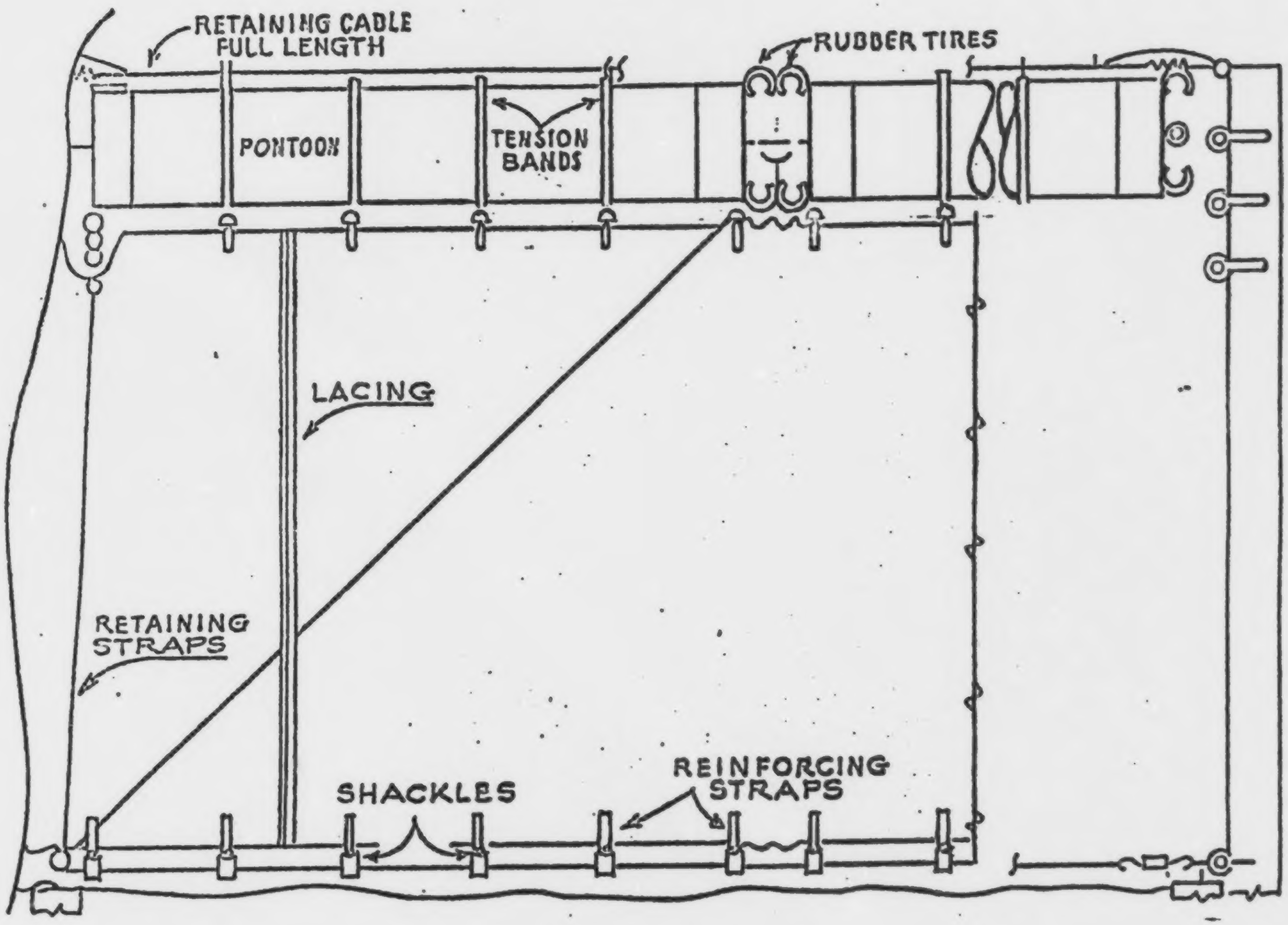

RETAINING CABLE FULL LENGTH
RUBBER TIRES
PONTOON
TENSION BANDS
LACING
RETAINING STRAPS
SHACKLES
REINFORCING STRAPS

Figure 2.

White City barrier.

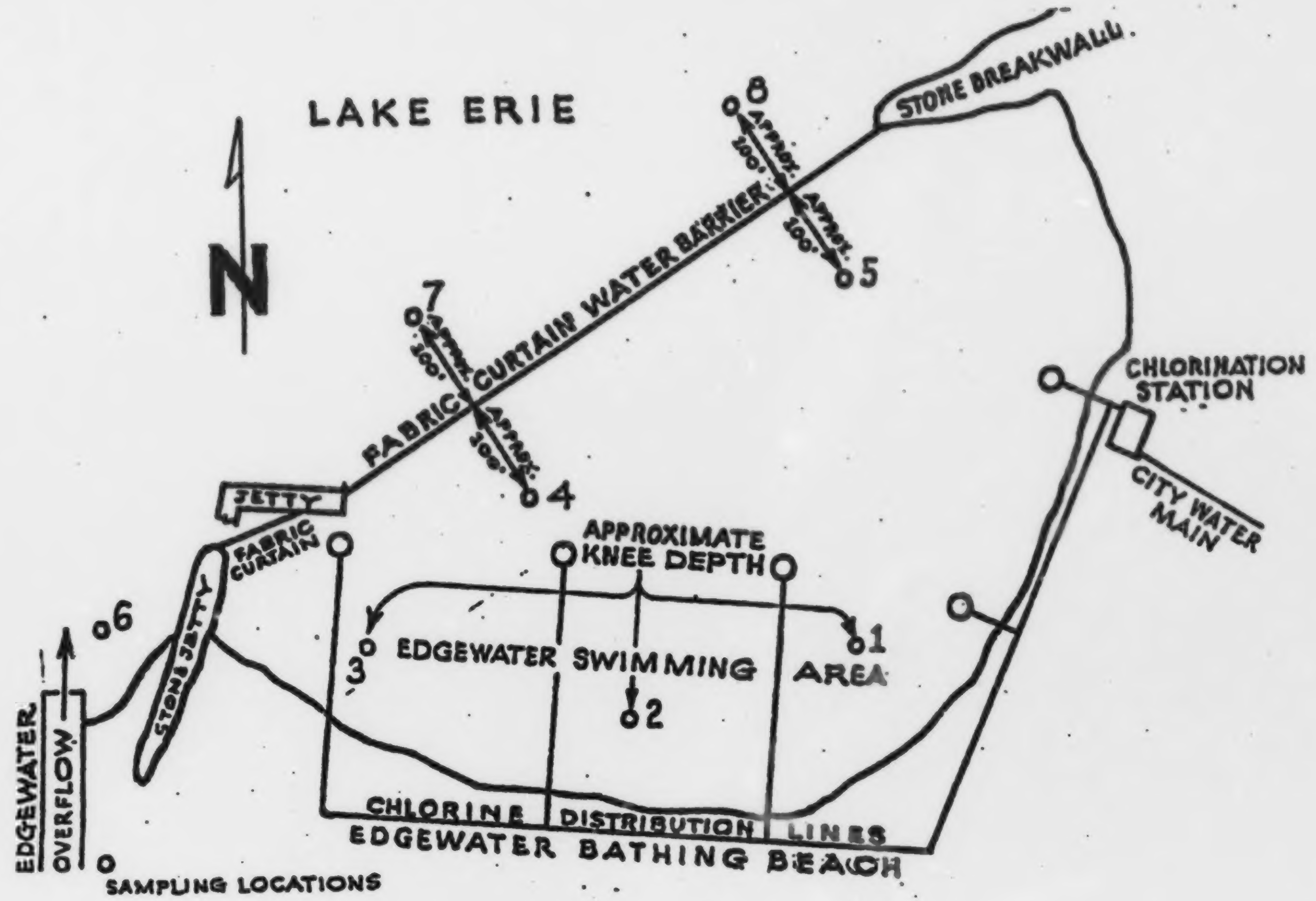
BARRIER AT EDGEWATER BEACH

Success with the White City Beach barrier in 1968 and a positive reception by the public encouraged a similar program at Edgewater Beach Park, on the City's west side. Edgewater Beach has long been a favorite recreation area to Clevelanders but attendance in recent years has been discouraged by unsightly debris and warnings of polluted water.

Although not as protected as White City Beach, the Edgewater bathing area is situated within a bay-like portion of the shoreline. Being directly exposed to all north and west winds, the barrier at Edgewater Beach had to be sufficiently durable to withstand up to eight foot waves. Preformed Line Products Company, Marine Systems Division, was contracted to design and construct a durable fabric curtain to be positioned across the mouth of the bay, and span two stone wall jetties which extend into the lake on both sides of the bathing area (Figure 3).

Thirty-two nylon fabric panels (Figure 4), twenty-five feet long were constructed. Into the top of each panel was sewn, as in a hem, six blocks of styrofoam seventeen inches square and four feet long. To reduce the additional installation expense of dredging the swimming area, the panels were cut to correspond to the various depths from the most shallow measurement of six feet to the deepest of fourteen. The panels were laced together with nylon rope and installed in sections. Sixteen hundred feet of forty-two pound anchor chain was laced in double fashion to provide anchorage for the 800 foot barrier. Attachments to the anchors were accomplished with heavy nylon straps wrapped in plastic sleeves to protect the fabric from sand abrasion. The straps were sewn into the fabric the entire length of each panel section.

The design of the Edgewater fabric curtain was such that it would submerge under a high wave (Figure 5). The resilience of the Edgewater barrier was demonstrated on July 4, 1969. A thunderstorm accompanied by winds of 100 miles an hour drove the barrier well below the surface of the water. Small pleasure craft, which were anchored off Edgewater Beach for a fireworks display, came over the barrier to seek refuge inside the beach area. After the storm subsided, the barrier returned to its original position, sustaining only two broken anchor straps. The Edgewater Beach barrier flotation was more continuous in construction than the White City Beach barrier, which had open sections between pontoons.



LAKE ERIE
N
STONE BREAKWALL
FABRIC CURTAIN WATER BARRIER
APPROX. 100'
APPROX. 100'
APPROX. 100'
APPROX. 100'
8
5
7
4
JETTY
FABRIC CURTAIN
STONE JETTY
6
EDGEWATER OVERFLOW
CHLORINATION STATION
CITY WATER MAIN
APPROXIMATE KNEE DEPTH
1
2
3
EDGEWATER SWIMMING AREA
CHLORINE DISTRIBUTION LINES
EDGEWATER BATHING BEACH
o SAMPLING LOCATIONS

Figure 3.

Edgewater Beach swimming area.

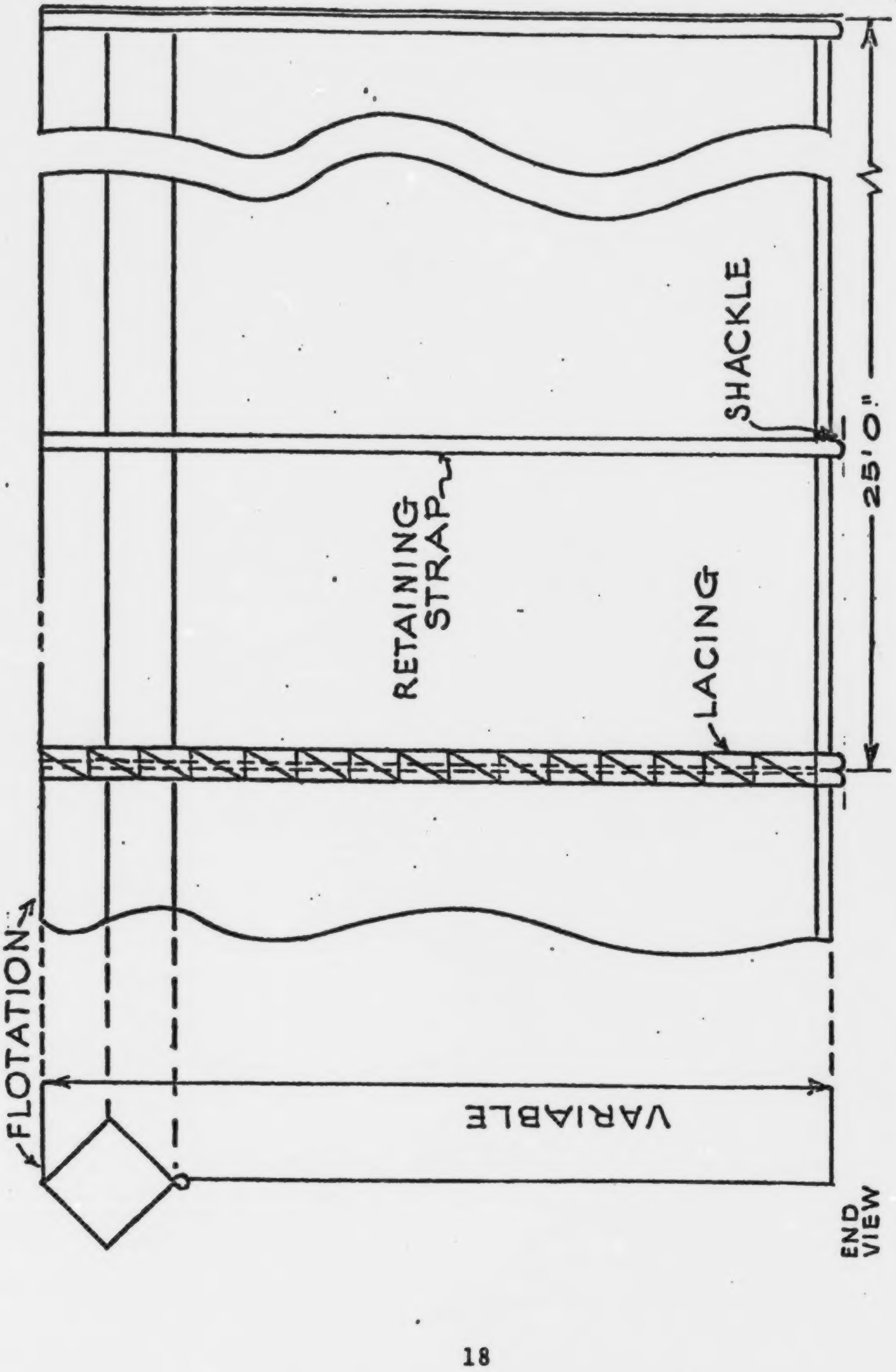

FLOTATION
VARIABLE
END VIEW
LACING
RETAINING STRAP
SHACKLE
25'0"

Figure 4.

Edgewater barrier.

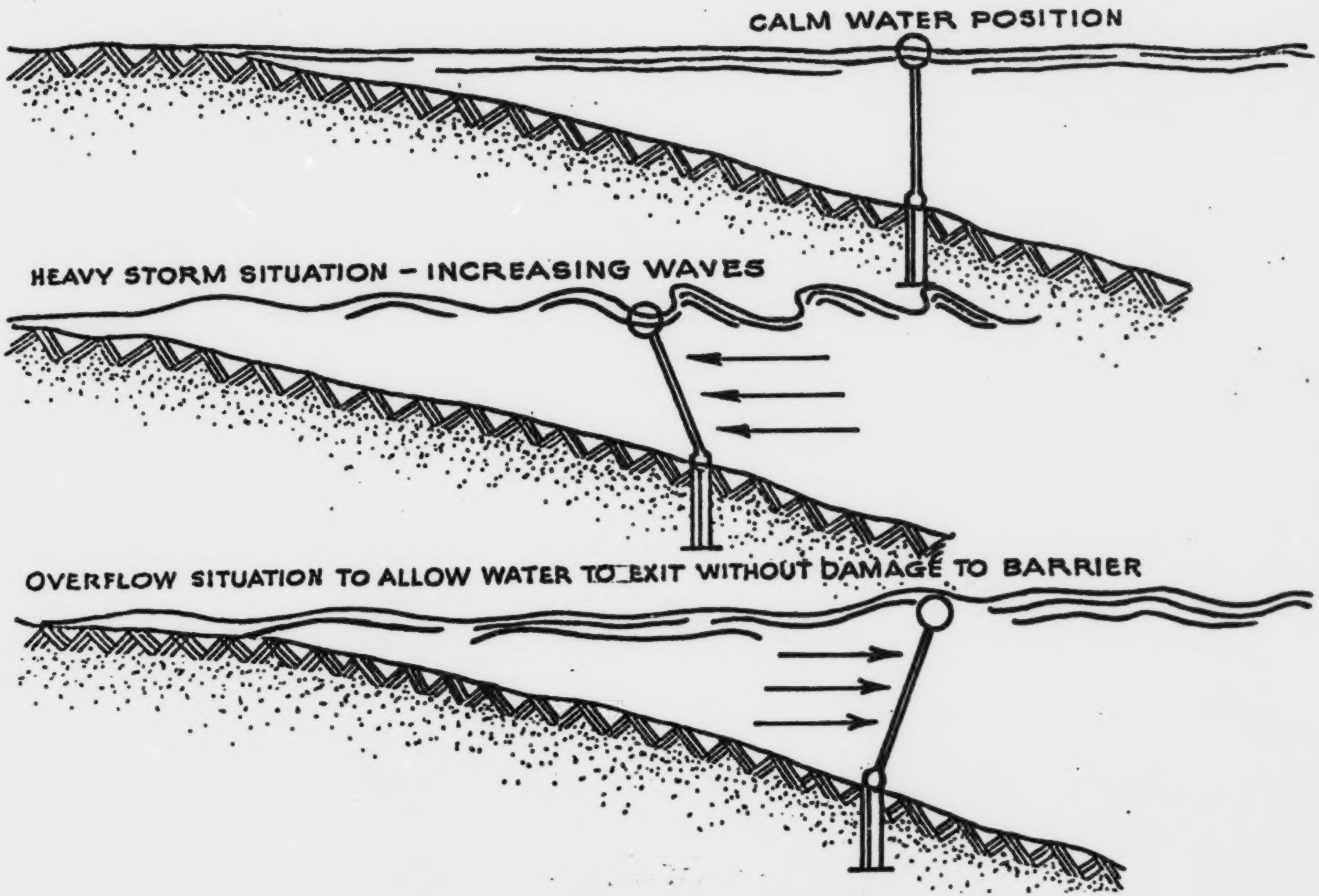

CALM WATER POSITION
HEAVY STORM SITUATION – INCREASING WAVES
OVERFLOW SITUATION TO ALLOW WATER TO EXIT WITHOUT DAMAGE TO BARRIER

Figure 5.

Effect of wave patterns on the barrier.

The Edgewater barrier did not only adapt well to isolating the enclosed area, but was also very effective in preventing the infiltration of floating debris. As a result, the debris on the beach was primarily limited to litter from bathers.

Except for occasional vandalism, the barrier suffered little damage during its first season in use. In 1969, the abrasive action of the sand on the nylon anchor straps caused some of them to wear through. At first, these anchor straps were replaced by one-quarter inch wire rope shackles. However, the shackles wore out within two weeks of installation. Replacement of all anchor straps in 1970 with nylon straps inside a plastic sleeve proved to be the most durable of all bindings used. Methods employed by the contractor to install the Edgewater barrier in 1970 were incorrect and on two occasions the barrier washed out and onto the beach before it could be anchored. After repairs were made on the barrier, it was correctly installed in three days.

DISINFECTION OF THE BEACHWATER

The State of Ohio requirements for safe bathing water are: No more than 200 fecal coliforms per 100 ml based on a monthly geometric means and not more than ten percent of the samples can have fecal coliform numbers greater than 400 per 100 ml. To meet the State of Ohio requirements for safe swimming, the beachwater had to be disinfected. Sodium hypochlorite, $NaOCl$, was chosen as the disinfectant for safety, availability, and potent germicide considerations.

DISINFECTION AT WHITE CITY BEACH

After the fabric barrier was installed, water within the enclosed area was displaced by pumping better quality lake water into the bathing area from approximately 1000 feet offshore. This was accomplished with a seventy-five horsepower centrifugal pump, pumping at a rate of 130,000 gallons per hour. At this rate, the entire volume within the enclosed area was displaced in three days.

A fifteen percent sodium hypochlorite solution was pumped from a four thousand gallon storage tank to the pump well of a centrifugal pump. There it was mixed with lake intake water before distribution through an eight inch pipeline to three points in the bathing area. Hypochlorite feed rates were adjusted twice daily according to residual chlorine analysis. Application rates varied from fifteen to twenty-five gallons of hypochlorite per hour.

The White City Beach area was sampled for bacterial analysis twice daily at 9:00 A.M. and 3:00 P.M. Five sampling points were selected within the enclosures and three outside. The densities of three pollution indicator bacteria were measured and all results were expressed in the geometric average number of bacteria per 100 ml of water.

Figures 6-8 show the results of the bacteriological profile. Prior to implementation and use of the curtain and disinfection, the beachwater was unfit for swimming. After the curtain was put into place and the hypochlorination facilities were working properly, bacteria counts in the beachwater area met with the State of Ohio health requirements.

Automatic chlorination used in conjunction with a fabric curtain barrier at White City Beach was shown to be a highly effective method of improving water quality for recreational purposes. Prior to chlorination and enclosure, the water quality was unsafe for human use. This is evidenced by a geometric average of 700 fecal coliforms per 100 ml of water, or 500 per 100 ml over the State of Ohio's limit. During periods of overflow, this number was increased to 30,000 fecal coliforms per 100 ml. During periods of enclosure and chlorination, the geometric average for fecal coliform bacteria varied from a high of 50 to a low of 6 per 100 ml during the swimming season. Outside the enclosed swimming area for the same period of time, the fecal coliform densities ranged from a high of 450 to a low of 260. The control points outside the barrier had a lower than normal bacteria count due to the diffusion of some chlorinated water into sampling sites adjacent to the barrier.

The fecal coliform density was controlled extremely well during periods of rainfall and combined sewer overflow. The geometric mean varied only from 4 inside the enclosure during dry weather to 10 during wet weather. Outside the enclosure, however, the fecal coliform density rose during wet weather to a high of 1200 per 100 ml.

High counts had occurred inside the barrier only when the level of residual chlorine inside the enclosure was undetectable. It is notable that when the chlorine residual inside the barrier approached zero, the fecal coliform density inside the enclosure was greater than control points outside. The geometric mean of fecal coliforms,

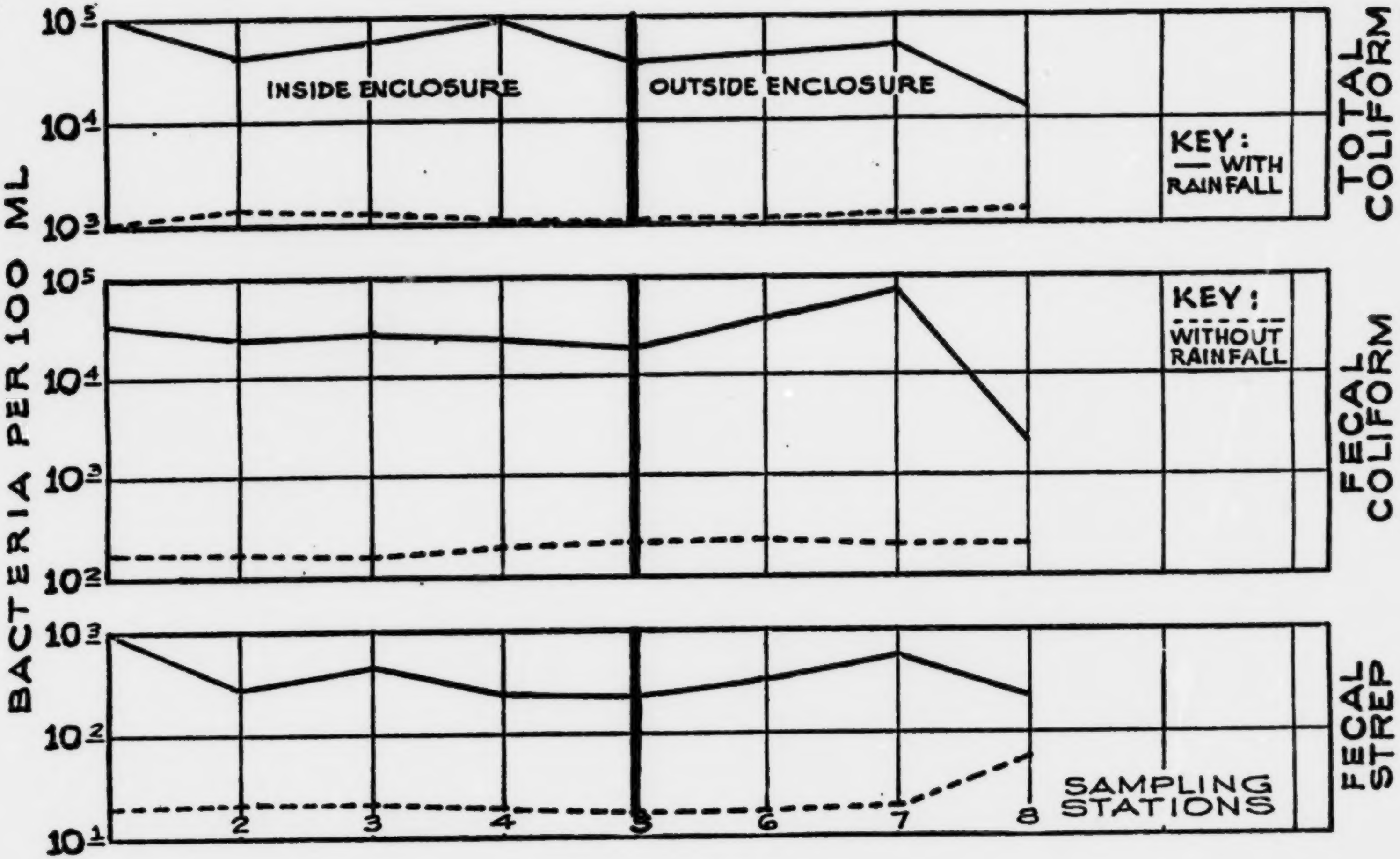


Figure 6.

Bacterial density profile at White City Beach prior to enclosure and chlorination 5/19 to 6/11

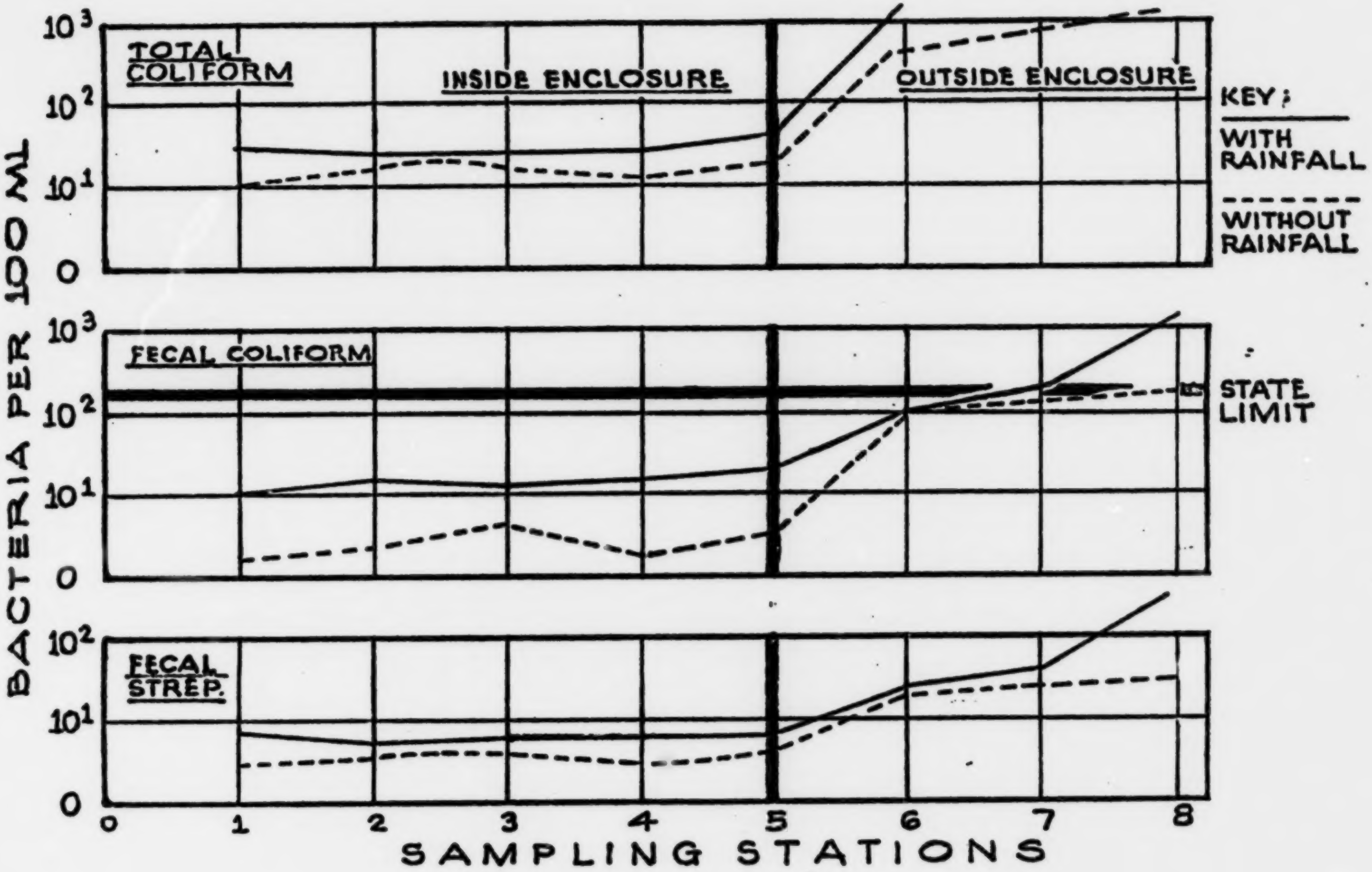


Figure 7.

Bacterial density profile at White City Beach during period of enclosure and chlorination 6/14 to 9/7

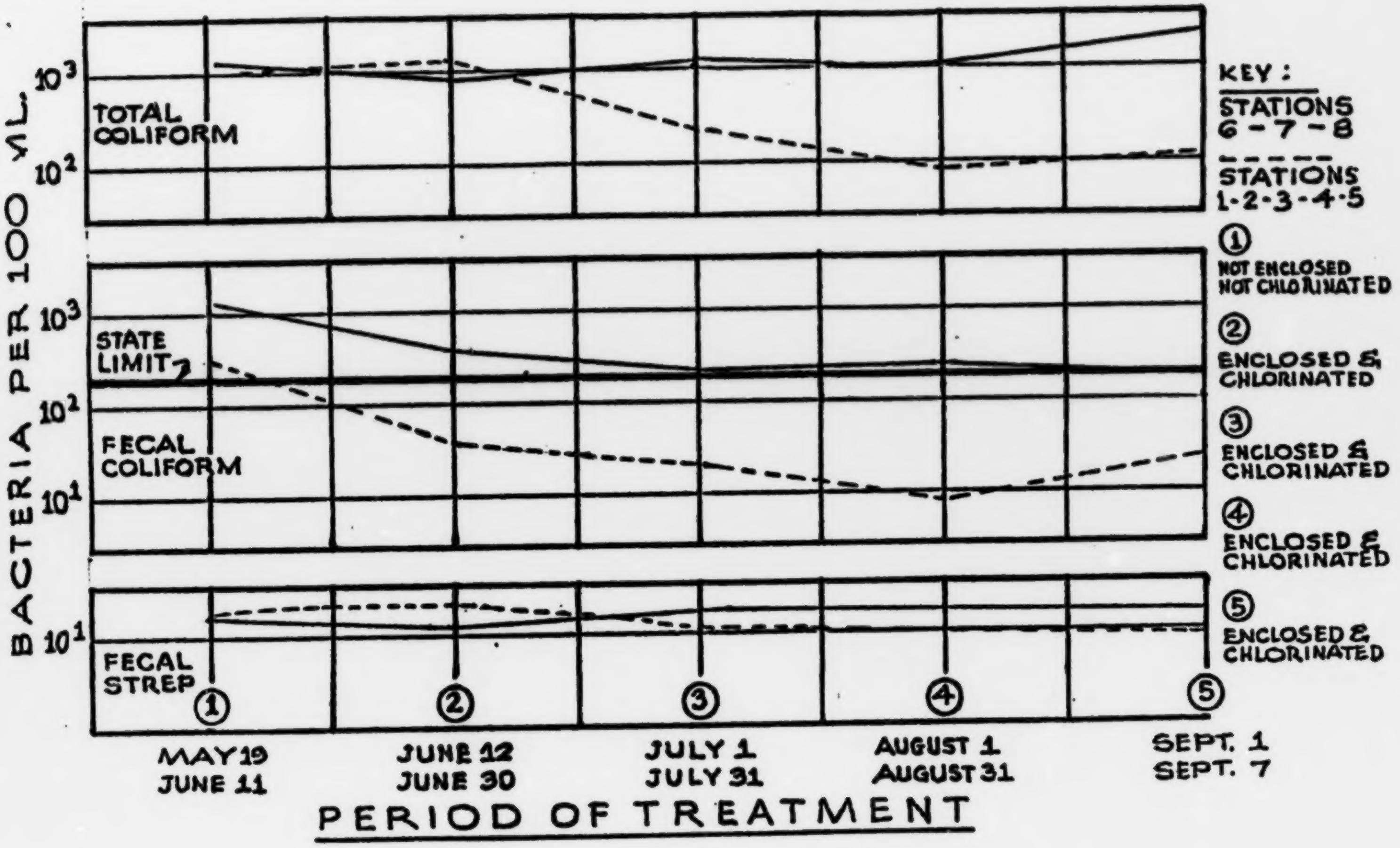


Figure 8.

Bacterial density profile at White City Beach
during the entire 1970 bathing season 5/19 to 9/7

during the zero chlorine residual period, was 80 bacteria per 100 ml inside the barrier while outside the density was 60 bacteria per 100 ml. This increase in bacteria can be attributed to the aftergrowth phenomenon.

DISINFECTION AT EDGEWATER BEACH

Disinfection of the waters of Edgewater Beach followed basically the same procedures as in the disinfection of White City Beach. At Edgewater, however, City tap water was used for the diluent for the sodium hypochlorite and as the source of fresh water to displace water in the beach area. A six inch pipeline delivered the tap water to the bathing area to approximately forty thousand gallons per hour. At this rate of flow, the entire volume of the ten acre swimming area could be chlorinated and displaced in ten days.

The sodium hypochlorite was introduced in the same manner as at White City Beach. The rate of application varied from seventy-eight to one hundred fifty-six gallons of sodium hypochlorite per hour. The hypochlorite delivery rates were adjusted twice daily after a chlorine residual analysis. Delivery rates were adjusted to maintain a chlorine residual of 0.2 to 0.8 mg/1.

Samples for bacteriological assays were taken at eight locations at Edgewater. Samples were taken daily and analyzed by Bacteriologists of the Bureau of Wastewater Quality. Figures 9-12 show the results of these bacteriological profiles.

Prior to chlorination and enclosure, the geometric mean ranged from 300 to 900 fecal coliforms at stations within the proposed enclosure. During periods of rainfall, a high of 3500 per 100 ml was recorded.

During periods of chlorination without enclosure, the geometric average for fecal coliform was 70 for all stations within the proposed enclosure. The geometric mean was 90 outside the area to be enclosed. Diffusion of the hypochlorite solution into the control areas was responsible for the unusually low values. The hypochlorination of the beachwater kept the bacteria counts within tolerable limits even without the barrier.

During the period of chlorination and enclosure, July 29 to September 7, 1970, the geometric average was 22 fecal coliforms per 100 ml and 14.8% of the samples exceeded the 400

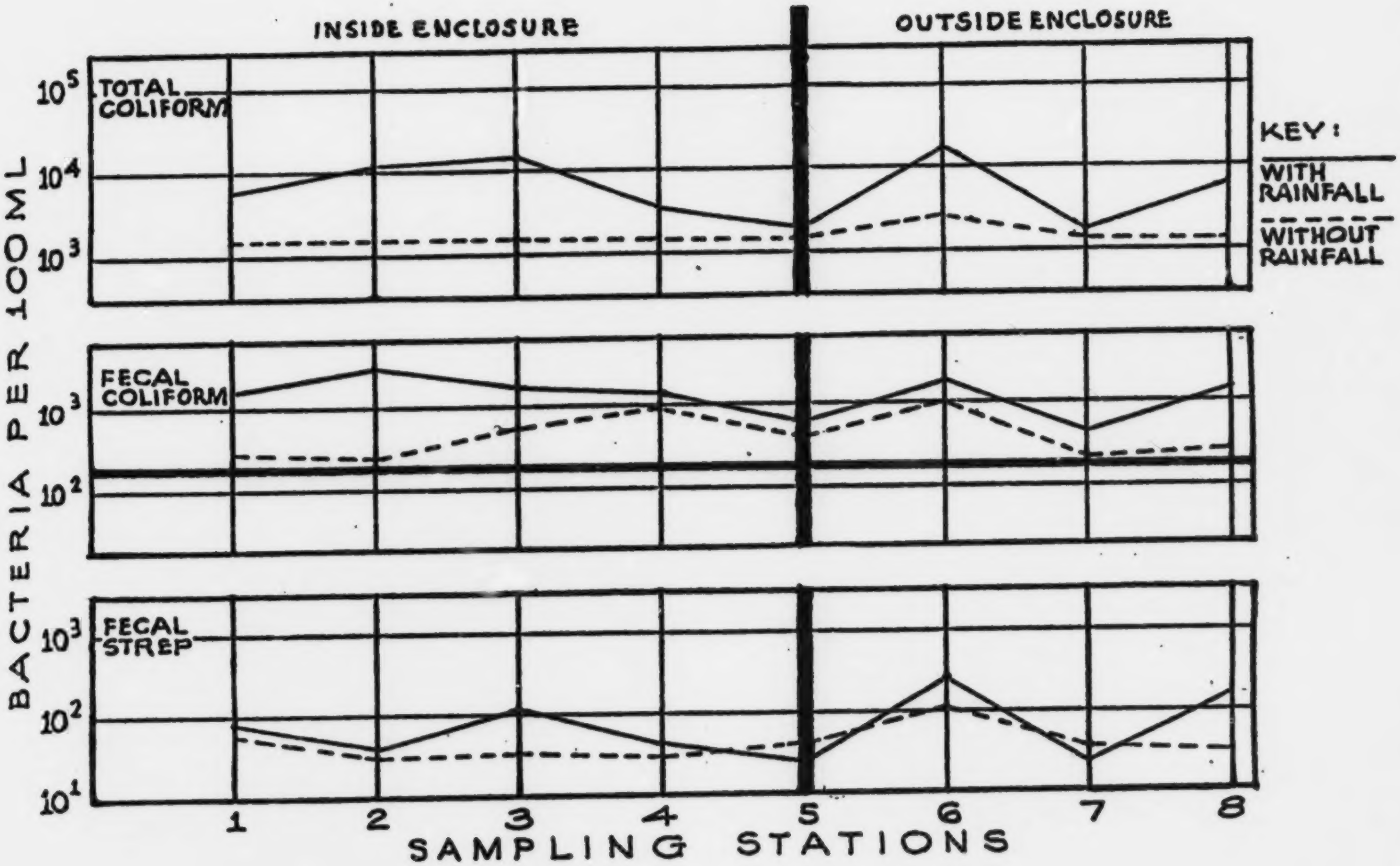


Figure 9.

Bacterial density profile at Edgewater Beach prior to enclosure and chlorination 5/19/70 to 6/13/70

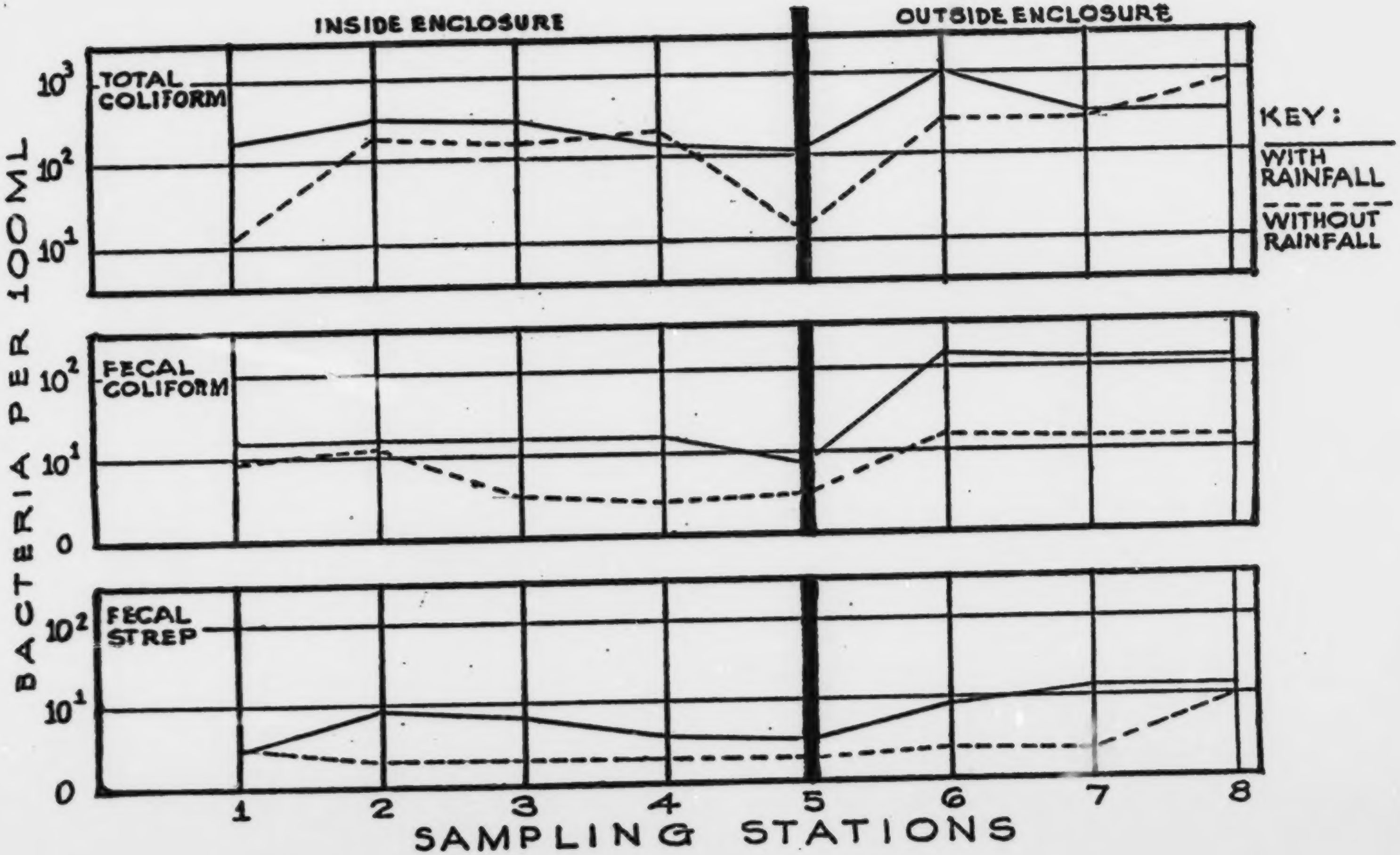


Figure 10.

Bacterial density profile at Edgewater Beach prior to enclosure but with chlorination 6/14/70 to 7/28/70

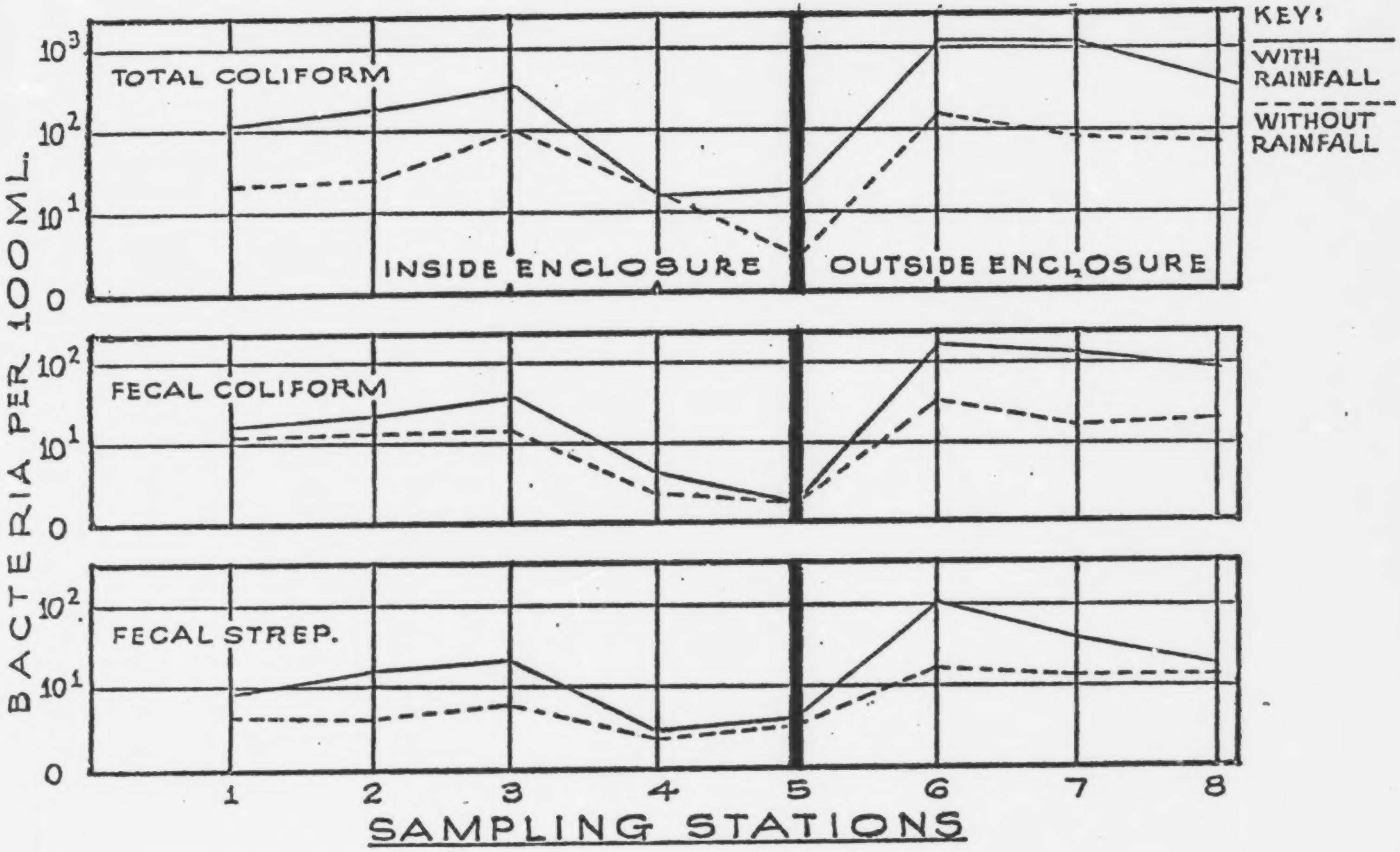


Figure 11.

Edgewater Beach during enclosure
and chlorination 7/29/70 to 9/7/70

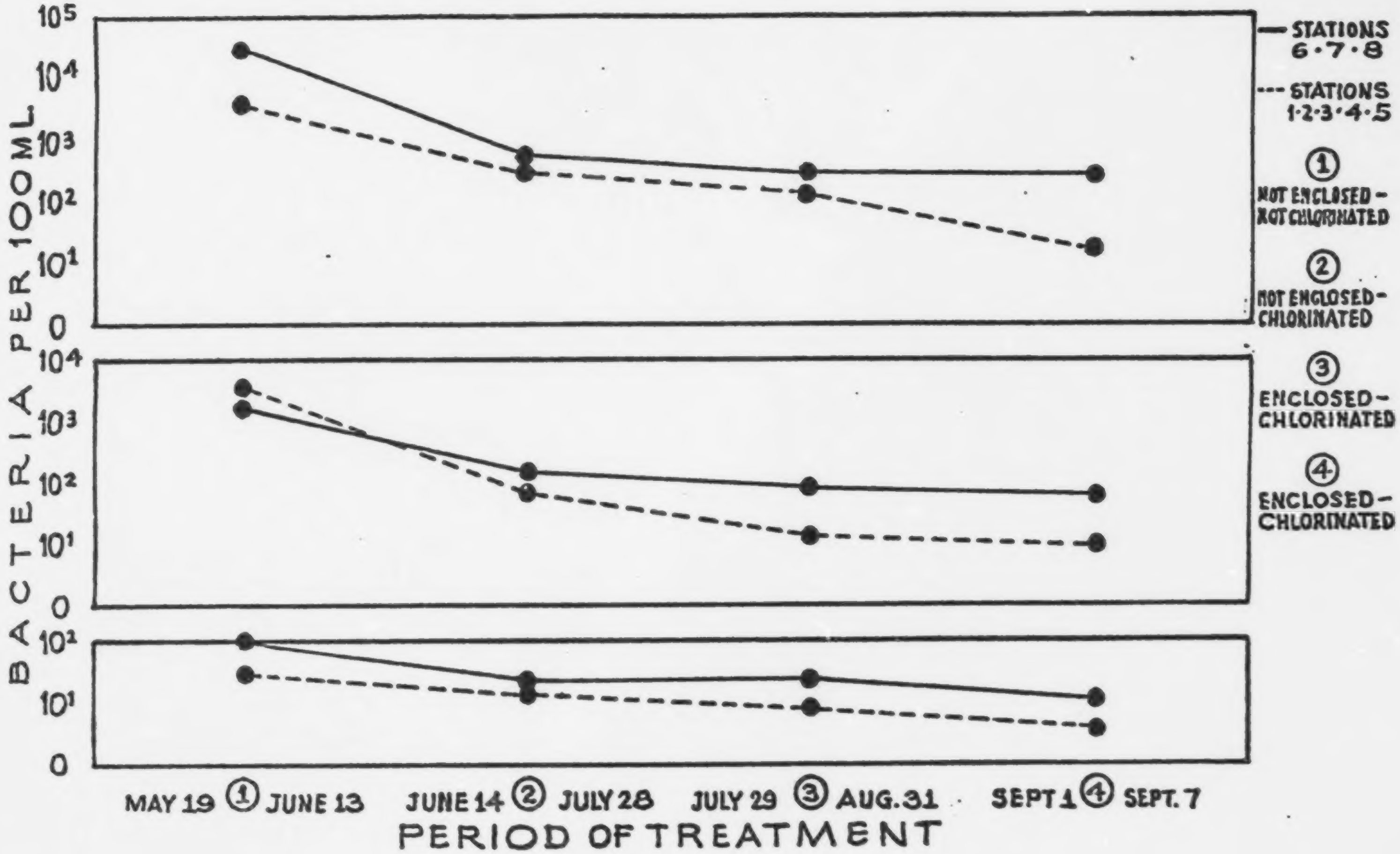


Figure 12.

Bacterial density profile at Edgewater Beach
during entire sampling period 7/29/70 to 9/7/70

per 100 limit. Outside of the enclosure, the geometric mean was 106, while 23% of the samples exceeded 400 ml per 100 ml. High bacteria counts at Edgewater Beach (during the enclosed period) occurred when the residual of chlorine was undetectable.

BEACH SANITIZING

A Beach Sanitizer was purchased in 1969 to aid in the debris removal from the beaches. This equipment aerated and sifted the sand as it removed debris from the beaches. The Beach Sanitizer was used alternately between the two beaches. Details of the Sanitizer are given in Section X. Figures 13-15 show the progress of the beach sanitizing effort.

DISCUSSION

The "Pool in the Lake" concept for providing a healthy, aesthetically pleasing swimming area has proven to be very effective. The beach project was successful in providing a safe recreational area in spite of the many problems that beset it.

The location of high volume storm overflows was unfortunate. White City Beach is located adjacent to a huge sewage plant bypass and combined sewage overflow conduit. This concrete conduit had cracks in it, allowing for infiltration of sewage into the beachwater. Edgewater Beach is located adjacent to a large combined sewer and storm overflow. These overflows were the major source of beach contamination.

Problems hindered the maintenance of chlorine residual. Disinfection by hypochlorination was stopped a number of times when the large volume used exhausted the manufacturers supply and when the caustic hypochlorite caused feeder pumps to fail. An equal dispersion of hypochlorite was difficult to maintain and often water at the shoreline received inadequate doses. Frequent high seas would swell over the barrier dissipating the chlorine residual. The barriers would then entrap the churned up turbid water that would wash over the barriers and hold it even after the lake would calm and clear.

There was a great deal of vandalism at the beaches. At the White City Beach, the PVC pipe that delivered the hypochlorite was ripped out of the ground and taken. On several occasions the curtains were slashed with knives. The swimming area at White City Beach provided a convenient

FIGURE 13

EDGEWATER BEACH PRIOR TO BEACH RECLAMATION

FIGURE 14

EDGEWATER BEACH DURING INITIAL STAGE

FIGURE 15

EDGEWATER BEACH WITH PROJECT WELL UNDER WAY

disposal point for stolen cars. Equipment for the preparation of the beaches in the spring had to include a crane for the removal of the cars. In a one week period in 1969, five cars had to be removed.

The anchorage was buried in the sand every winter. The enclosure at White City had to be dragged in the spring to uncover the anchoring points.

The beach sanitizer was effective in cleaning the beaches; however, it uncovered the moist sand substrate. After cleaning, the beach sand tended to be hard and caked.

There was a large fish kill (Appendix B) associated with the hypochlorination of the beachwater.

SECTION V

HYPOCHLORINATION OF STREAMS AND OVERFLOWS

INTRODUCTION

The success of the Beachwater clean-up of White City and Edgewater Beaches would be extremely tenuous if the waters surrounding the enclosed bathing area maintained a large amount of pollution and bacteria. In an attempt to reduce the number of bacteria in waters near the swimming areas, streams and combined sewer overflows were hypochlorinated.

Seven hypochlorination stations (Appendix C) were built at outfalls of heavily polluted water that directly affected the quality of the swimming area. The location of these hypochlorination stations are shown in Figure 16. Sodium hypochlorite was used because of the same considerations as with beach disinfection. The hypochlorination stations were designed to operate automatically. Since the stations are in populated areas and would be unattended, chlorine gas would have been too hazardous. Sampling points for determining the chemical and bacteriological characteristics were selected at points above and below the hypochlorination facility and along the lakefront. Samples were collected both prior to and after hypochlorination to gauge the effectiveness of the disinfection.

Dugway Brook, Shaw Brook, and Nine Mile Creek were the streams chosen for the study. These partially culverted and grossly polluted streams receive large quantities of sanitary overflow and industrial discharge. These streams were treated with sodium hypochlorite from June 10 to August 29 during the 1969 season and from June 14 to September 7 during 1970. The other four stations, West 117th, Edgewater, Westerly and Easterly combined sewer overflows received doses of hypochlorite only during periods of overflow. Different types of hypochlorination equipment were used at the various sites in an effort to determine which types of equipment are most satisfactory (Appendix C).

METHODS

Electronic metering and feeding equipment gauged the flow of the streams and overflows and fed sodium hypochlorite proportionally. The rate of sodium hypochlorite fed into the streams and at the overflows was adjusted to produce a 3 mg/l chlorine residual 15 minutes downstream of the hypochlorinators. Samples for bacteriological and chlorine residual analyses were collected on a seven day per week basis and twice daily during periods of wet weather. Wet weather in this study is defined as a day of rainfall and two subsequent days. The two sampling points located at the extreme West and East of the lakefront were not near a polluted outfall or a hypochlorination facility, therefore, samples taken from these points were used to yield comparative data. All bacteriological analyses were determined by the membrane filter technique according to Standard Methods for the Examination of Water and Wastewater, 12th edition, 1965. The data for each bacteriological analysis was fed into the City of Cleveland's IBM 360 computer. The computer was programmed to compile the data and calculate averages, standard deviation, and geometric means.

RESULTS

The following graphs (Figures 18-22) show the bacteriological results prior to and during hypochlorination.

As the graphs indicate, the hypochlorination of Shaw Brook, Nine Mile Creek, and Dugway Brook was not effective in reducing the number of bacteria being discharged into Lake Erie. The hypochlorination of storm overflows, however, did significantly reduce the bacteria count.

Hypochlorination of the streams did reduce the bacteria levels but the reduction occurred just below the hypochlorination stations. But, as the stream approached Lake Erie, the bacterial numbers increased logarithmically.

Table 1 summarizes the effectiveness of the hypochlorination in wet and dry weather during the hypochlorination phase of the program. The effectiveness of the disinfectant was significantly reduced during periods of wet weather with the attendent storm overflow of sanitary sewage.

DISCUSSION

The results of hypochlorination of streams and overflows show a definite reduction in pollution indicator bacteria at the overflows but little or no reduction of these bacteria in the streams at their point of entry into Lake Erie. Data from the west side sampling sites indicate that even though a large percentage of fecal coliform bacteria in overflows was killed during chlorination, the remaining number of these bacteria greatly exceeded water quality standards.

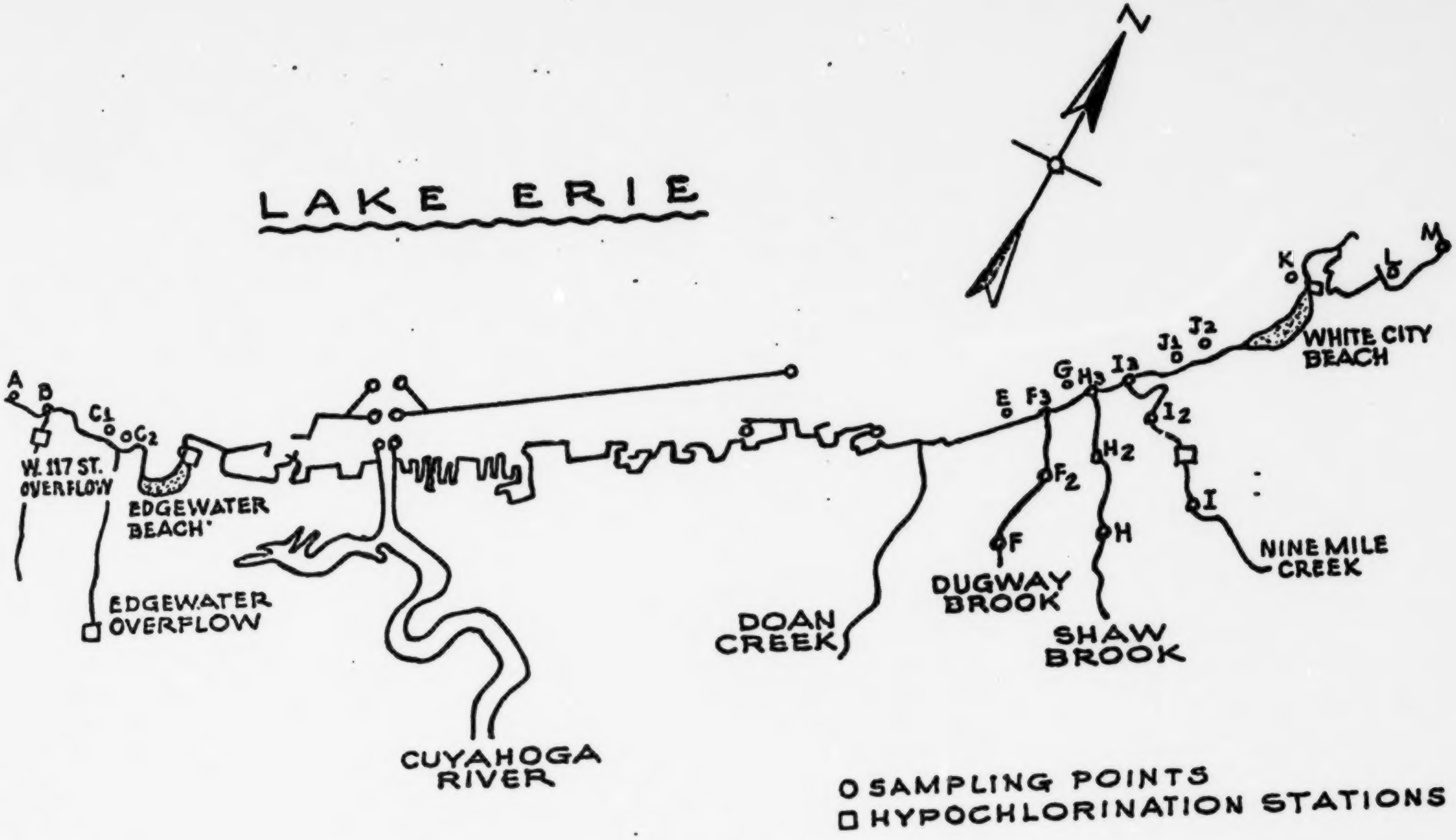

LAKE ERIE
N
A
B
C1
C2
W. 117 ST. OVERFLOW
EDGEWATER BEACH
EDGEWATER OVERFLOW
CUYAHOGA RIVER
DOAN CREEK
E
F3
F2
F
DUGWAY BROOK
G
H3
H2
H
SHAW BROOK
I3
I2
I
NINE MILE CREEK
J1
J2
K
L
M
WHITE CITY BEACH
○ SAMPLING POINTS
□ HYPOCHLORINATION STATIONS

Figure 16.

Locator map of the hypochlorination stations and sampling points

FIGURE 17

HYPOCHLORINATION FACILITY ON DUGWAY BROOK

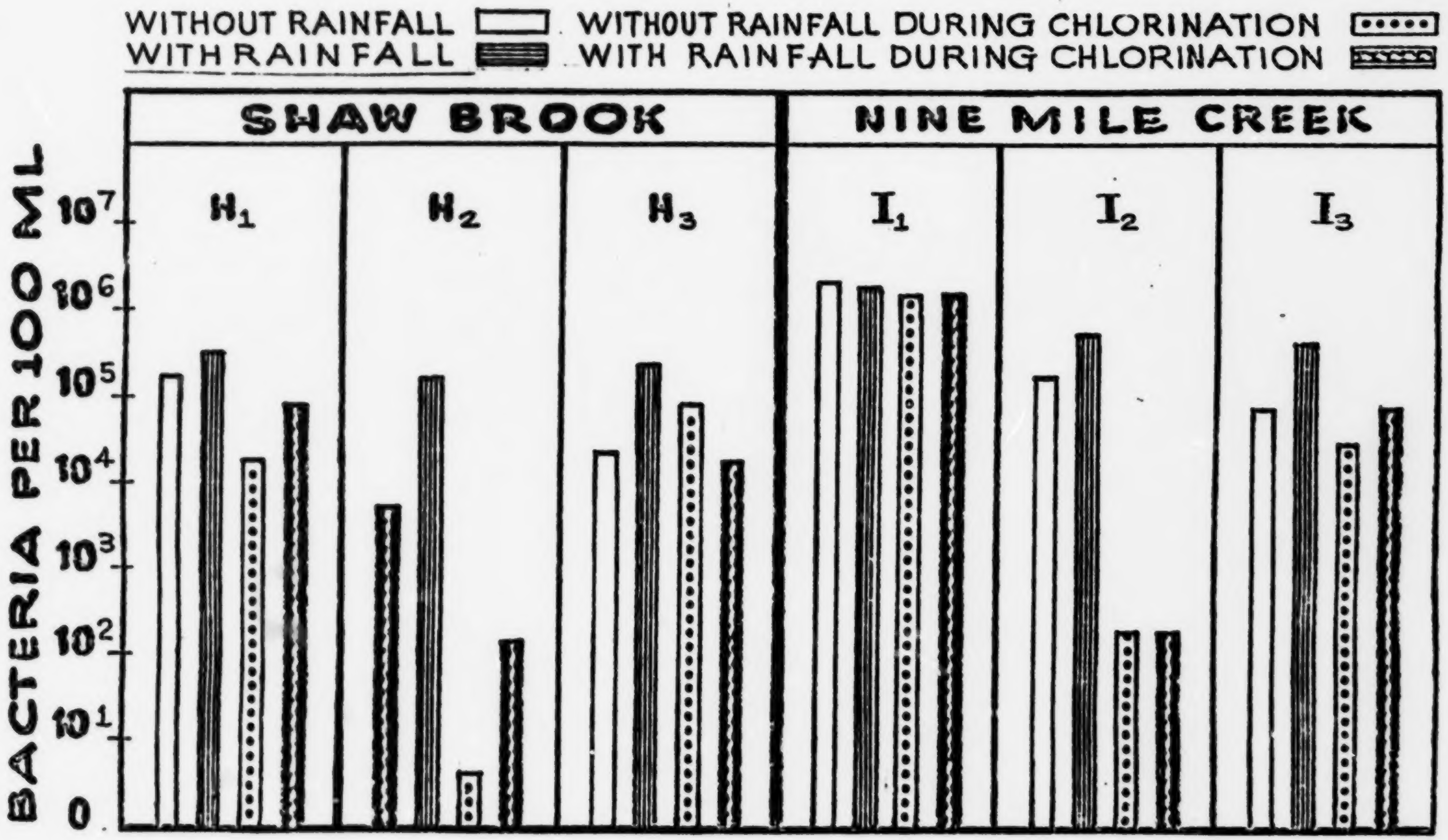


Figure 18.

Total coliform density
(prior to and during chlorination)
5/19/70 - 9/7/70

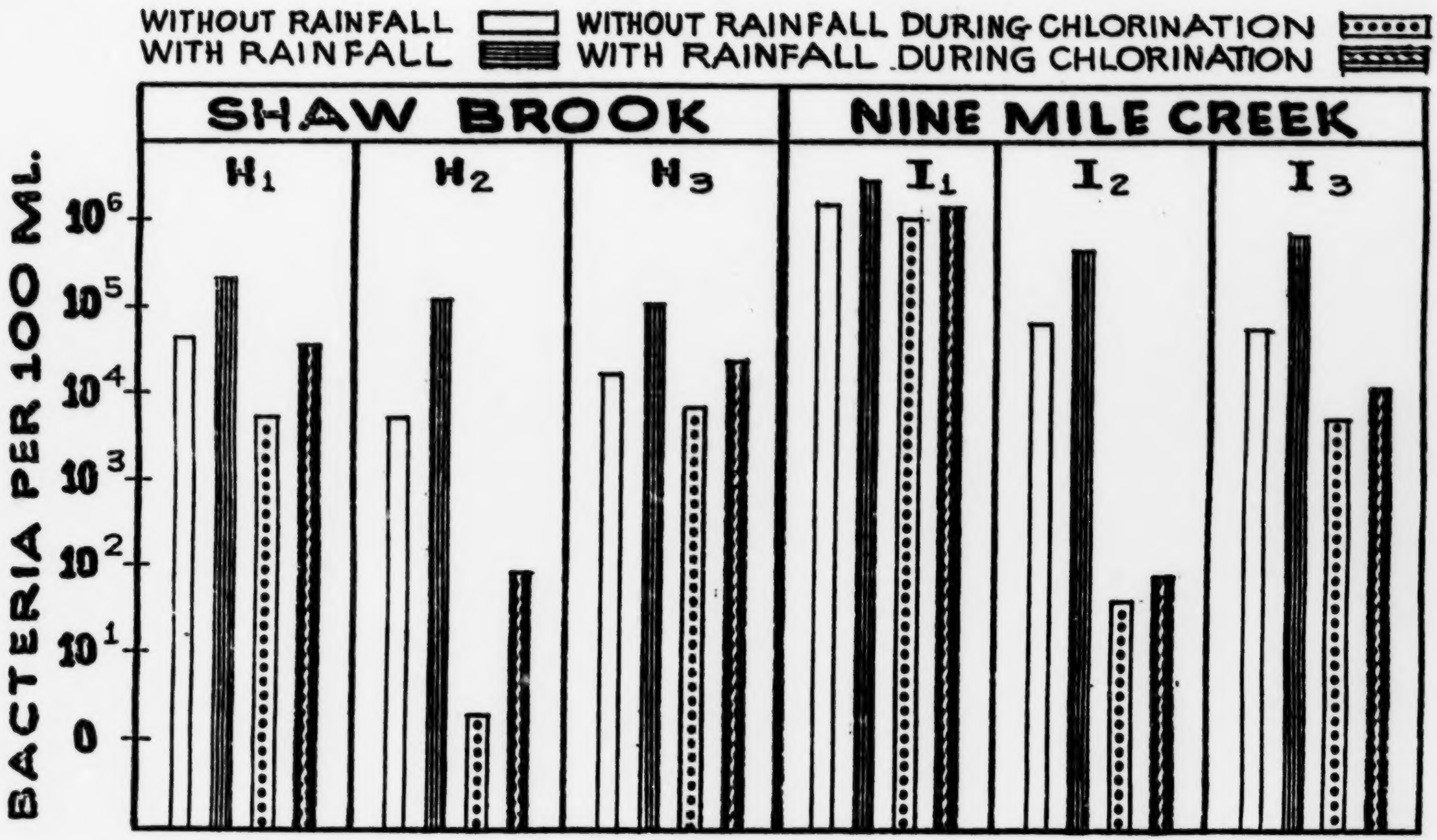


Figure 19.

Fecal coliform density
(prior to and during chlorination)
5/19/70 - 9/7/70

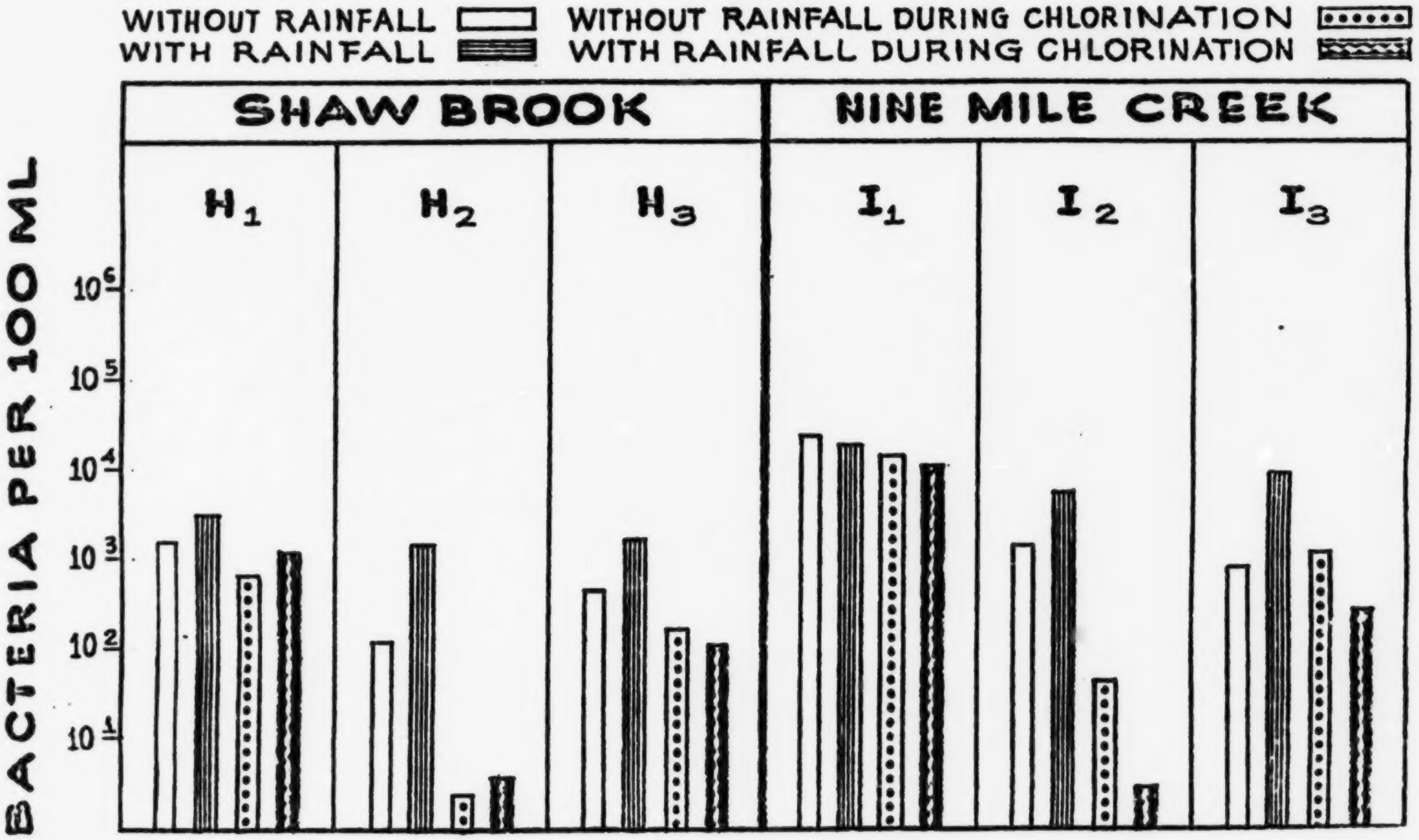


Figure 20.

Fecal streptococcus density
(prior to and during chlorination)
5/19/70 - 9/7/70

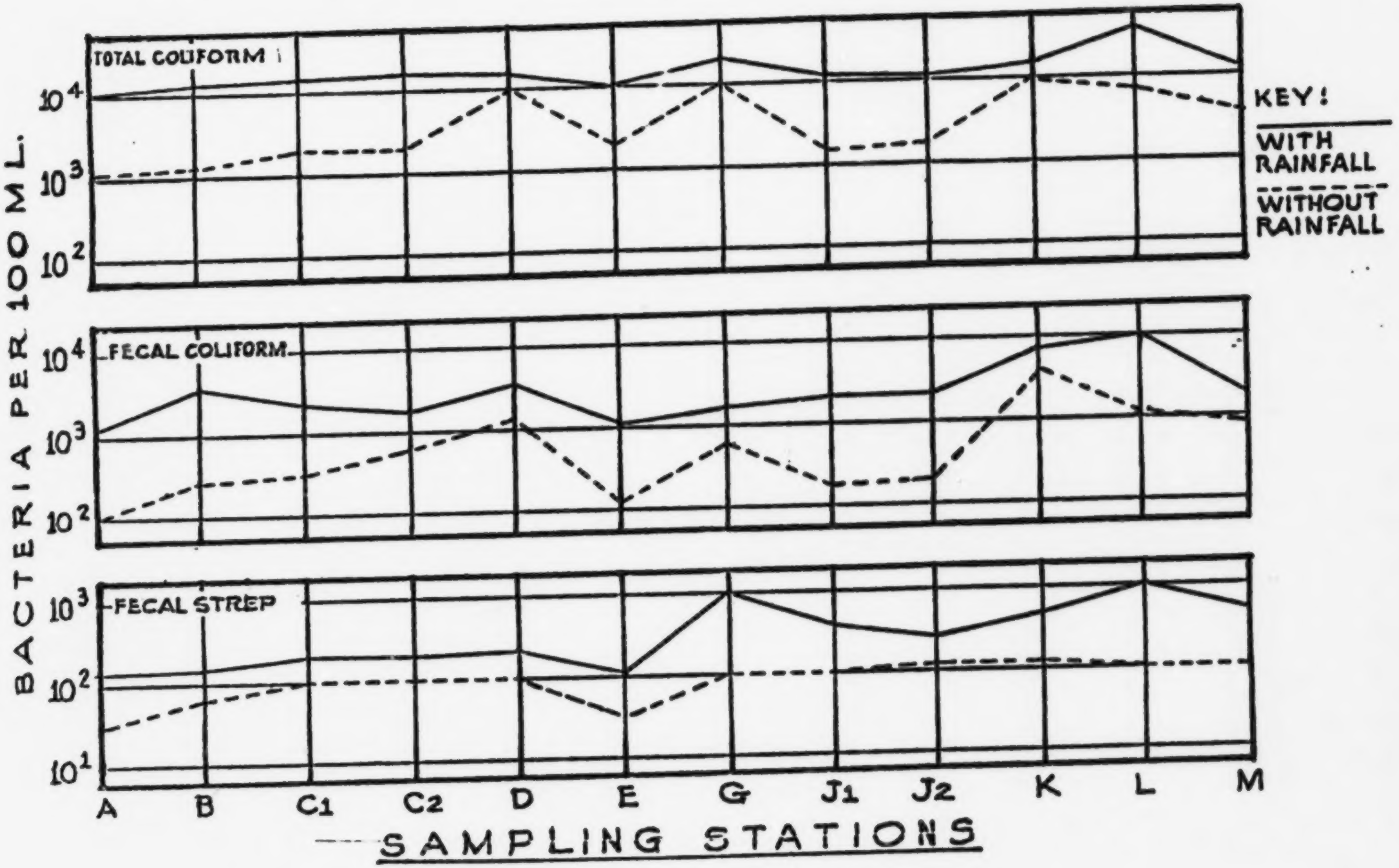


Figure 21.

Shoreline profile Stations A-M - Bacterial density before chlorination period 5/19/70 to 6/11/70

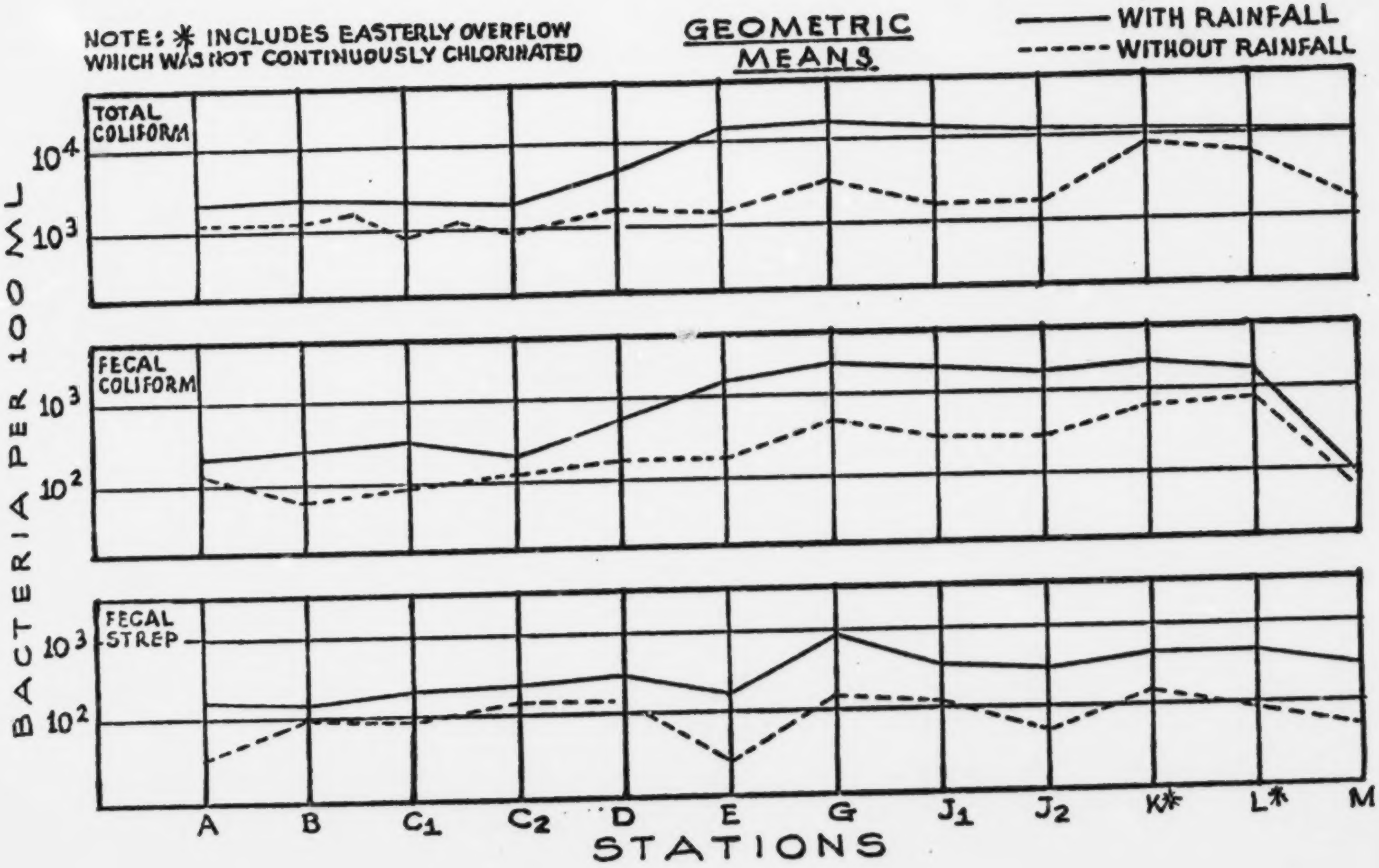


Figure 22.

Shoreline profile Stations A-M - Bacterial density of period of chlorination

Table 1. FECAL COLIFORM DENSITY DURING PERIOD OF CHLORINATION 1969 - 1970

Beaches	1969 Dry + Wet	1970 Dry	 Wet	Streams	1969 Dry + Wet	1970 Dry	 Wet
A		200	450	H_1	149,000	2,000	9,500
B	2,400	90	500	H_2	2,800	2	50
C_1		100	550	H_3		2,500	9,000
C_2		200	350	I_1	346,000	250,000	300,000
D	15,000	250	650	I_2	1,000	30	50
E	3,400	300	2000	I_3		2,500	5,000
G		750	3500				
J_1	3,600	500	2500				
J_2		450	2000				
M	10,000	70	90				

The hypochlorination facilities at the Edgewater Overflow were inoperative on several occasions and the resultant fecal coliform densities during these periods were as follows:

Date	Station	Fecal Coliform: No chlorination	Range during chlorination
6-26-70	C_1	11,000	
	C_2	46,000	100-150
7-10-70	C_1	21,000	200-250
	C_2	47,000	
7-21-70	C_1	1,000	
	C_2	1,600	

On these particular days, the fecal coliform density ranged from 100 to 47,000 per/100 ml. This range could be expected for the entire summer during overflow at these stations if no chlorination facilities were available. The fecal coliform density when the chlorination facilities were in operation ranged from 100 to 250 per/100 ml.

The lack of success in reducing bacteria along the lakefront by the hypochlorination of streams and overflows and the evidence of bacterial aftergrowth prompted an investigation of the phenomenon of bacterial aftergrowth.

SECTION VI

BACTERIAL AFTERGROWTH

INTRODUCTION

Upon the analysis of the bacterial data compiled in conjunction with the hypochlorination of streams and overflows (see Section V), it was noted that in some instances bacterial densities during periods of hypochlorination were higher along the lakefront than periods when hypochlorination was not in effect. It was suspected that even though hypochlorination initially resulted in substantial reduction in the numbers of bacteria, as soon as the hypochlorite dissipated, the remaining viable bacteria were able to utilize the nutrients in the streams and entered into a logarithmic growth phase.

Subsequent laboratory bottle-tests using raw sewage, sewage treatment plant effluent, storm water, and a stream sample showed that the bacteria did, indeed, enter into a growth phase following chlorination (see Appendix E). In most instances, the aftergrowth resulted in higher bacterial densities than were present prior to chlorination.

Other investigators have reported similar results. Evans et al[1] found that aftergrowth of three indicator bacteria groups occurred in chlorinated stormwater. In their study, the peak aftergrowth for fecal coliform and fecal streptococcus was inversely related to the chlorine dosage applied. Weibel et al[2] showed evidence of aftergrowth using settled storm water. Heukelekion[3] found the coliform aftergrowth occurred and that aftergrowth was directly related to the concentration of the chlorinated wastewater present.

Since disinfection by chlorination was used extensively as an interim technique for reclamation of beachwater and it has been shown that bacterial aftergrowth negates its effectiveness, the Bureau of Wastewater Quality of the City of Cleveland conducted a research project to try to determine the cause of and possible means to reduce aftergrowth. The research of the aftergrowth phenomenon was divided into two phases: 1) An *in situ* stream study of bacterial aftergrowth resulting from hypochlorination, and 2) bench scale studies on possible relationship between aftergrowth and chlorination due to chlorination-induced changes.

The intent and objectives of this aftergrowth study were to:

1. Determine to what extent bacterial aftergrowth occurred.
2. Determine whether hypochlorite input measureably affects any resultant aftergrowth.
3. Determine the effects of chlorination upon the chemical environment.
4. Determine if any relationship exists between aftergrowth and particle size distribution.
5. Determine if aftergrowth occurring *in situ* are from sources other than the treated stream waters, and
6. Determine if aftergrowth of indicator bacteria signifies a corresponding aftergrowth of pathogenic organisms.

IN SITU STREAM STUDY

Dugway Brook discharges into Lake Erie at Bratenahl, Ohio. The Brook is formed by the confluence of two branches just south of Lakeshore Boulevard. Its drainage area at the mouth is 5600 acres. Near its headwaters, it serves as a storm water receptor for several suburban communities. Where the two branches pass through Cleveland they are entirely culverted, and receive numerous sanitary discharges. As a result, the dry weather flow is predominantly raw sewage and ground water infiltration. At the point where the two culverted branches of the brook come together, there is a weir which serves as a chlorine contact basin.

Below the weir, the stream channel is open and winds through a wooded section for 2200 feet. In this stretch, the stream averages sixty-five feet in width and two to four feet in depth. This section does not receive any sanitary discharge, but does receive flow from two minor streams and three ground water discharges which flow only during periods of extreme wet weather. There is a sixth occasional discharge from a swimming pool in an adjacent apartment complex. The bottom of this stream was sand and gravel with no mud or sludge deposits. A marked increase in bacterial population would not, therefore, be caused by other downstream sources.

The flow characteristics of Dugway Brook and its dispersion in the lake were determined by a dye study (Appendix D). Two parameters were measured: (1) Time of travel, and (2) dilution ratios of the stream discharge by lake water. Both parameters were measured over the distance between the hypochlorination and points out into the lake.

From the hypochlorination station to 500 feet downstream, the travel time was 30 minutes. At 500 feet downstream of the hypochlorination facility, the stream became broader and deeper, slowing the flow to approximately 105 feet per hour from that point to the mouth of the stream. Dilution of the stream by lake water began approximately 2000 feet downstream of the hypochlorination facilities. At 2100 feet, mixing of the two waters was considerable and varied, the percent of stream water ranging from 10% to 36%. At 2200 feet the percent of stream water varied from 5% to 36%. The wide variances in the dilution ratios in the area of 2000 feet to 2200 feet downstream was caused by alternate surges of lake water into the stream.

In the lake, there was very little variance in the dilution ratio at any particular sampling point. At a point 200 feet east of the stream mouth, range in dilution values was 3% to 10%; at 500 feet east, 3% to 4.6%; and at 1100 feet east, 3% to 4.6%. There were no differences between the concentration of samples taken at the surface of the lake and those taken near the bottom. At any given distance from the mouth along the lakefront, the percentage of stream water present was the same at 25 feet from the lakefront as at 100 feet from the lakefront indicating that the Dugway discharge was well mixed with lake water.

Two thousand feet west of the stream mouth there was no detection of dye at any time. This point was used as a control station for comparing the effects of instream chlorination or bacterial growth in the lake.

During this study, several factors were investigated which included: the effects of chlorination, stream travel time, stream dilution, and difference levels of residual chlorine on bacterial aftergrowth. The study began on May 18, 1970, and continued through September 19, 1970. Samples were taken prior to chlorination from May 18 to June 25. Chlorination was started on June 26 and samples were taken from this date until September 19.

Throughout the two periods, samples for bacterial analysis were collected on a daily basis between 9:00 A.M. and 11:00 A.M.. The populations of total coliforms, fecal coliforms, and fecal streptococcus were routinely measured. These results are given as geometric means. Occasional samples from stations 1 and 4 were analyzed for Salmonella, an enteric pathogen. Samples for chemical analyses were collected on a bi-weekly schedule.

Sampling locations are shown graphically in Figure 23 and the locations are presented in Table 2. The sampling stations were chosen to cover a range of stream travel-times from the hypochlorination facility. A baseline station for the measurement of residual chlorine (Station 2) was established 200 feet downstream of the hypochlorination facility. The dry weather flow-time from the chlorinator to Station 2 was 15 minutes.

RESULTS - PRIOR TO CHLORINATION

A total of 363 samples were analyzed for bacteria during the period prior to chlorination. The mean level for all three groups of indicator bacteria showed a pattern of progressive decline from Station 1 to Station 5 at the mouth of the stream.

The level of bacteria at sampling stations in the lake varied in relation to the percent of stream water present, although the variations were not large. The population densities were lower than were expected on the basis of the percent of stream water present, however, at Station 9 (at the mouth of Dugway Brook), for example, the stream water concentration was nearly 50% of the concentration at Station 5, but the level of total coliforms was only 10% of the level at Station 5. The level of fecal coliforms and fecal streptococcus at Station 9 were only 1% and 5% respectively of the expected levels.

Comparisons were made between bacterial levels during dry weather periods and during wet weather periods. A wet weather period was defined as the day of any reported rainfall and two subsequent days. In general, bacteria levels in the stream decreased during wet weather periods. The opposite was true at the lake sampling stations. The general decreases in the stream were most likely due to dilution by storm water which contained proportionately fewer bacteria. Increases of bacteria in the lake during wet weather periods were the result of high concentration of stream water. Although the stream water had a smaller bacterial density during wet weather, the great increase in volume compensated for the lower density, thus giving higher bacterial concentrations along the lakefront.

Prior to chlorination of the stream, Salmonella was detected on each of three attempts to analyze for this organism. On each of the three occasions, samples were taken from Stations 1 and 4. Of the six total samples, only one was negative, and this occurred for a 50 ml sample from Station 4 (Table 3).

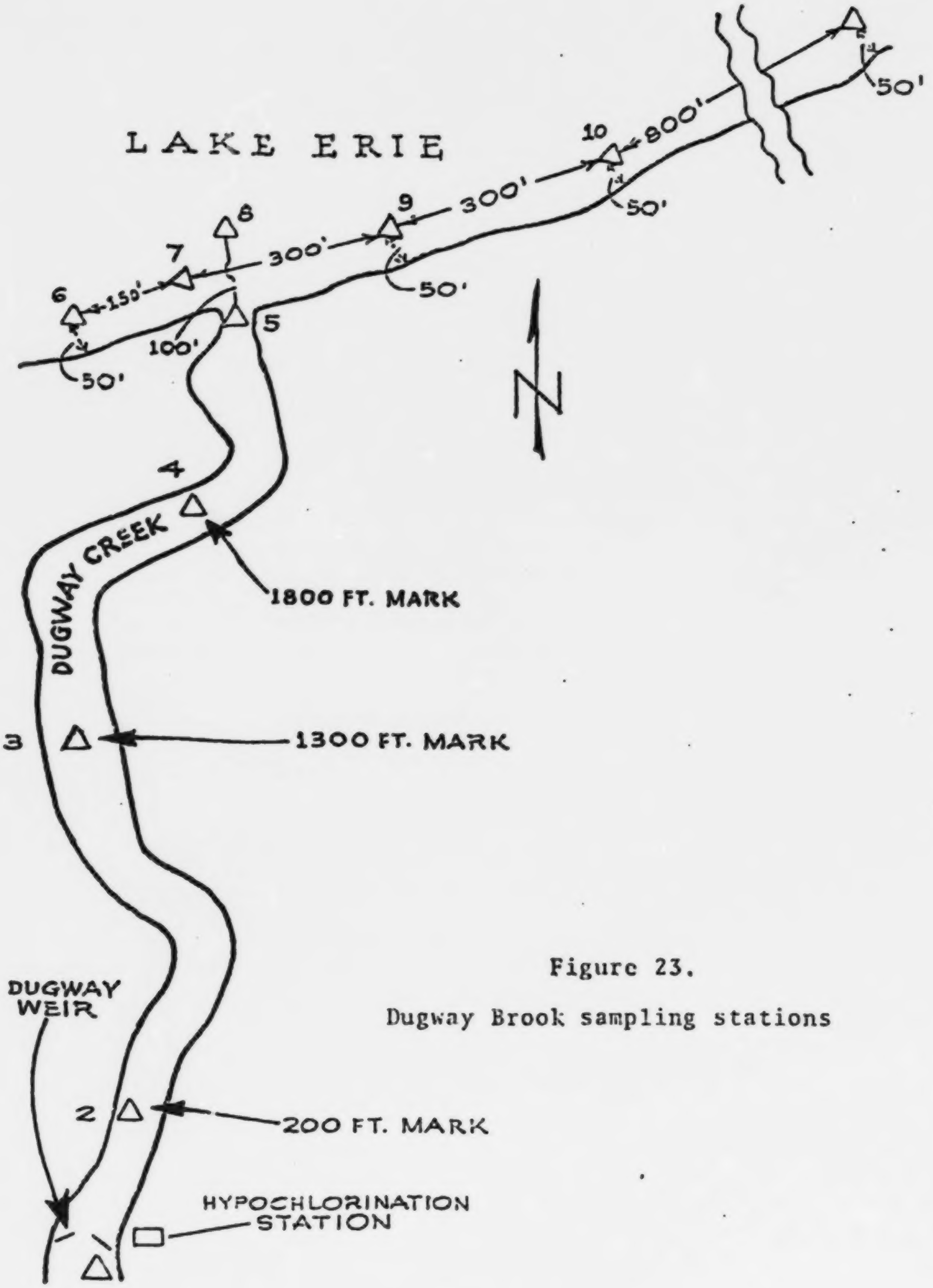

LAKE ERIE
10
800'
50'
9
300'
50'
8
7
300'
6
150'
50'
100'
5
50'
N
4
DUGWAY CREEK
1800 FT. MARK
3
1300 FT. MARK
DUGWAY WEIR
2
200 FT. MARK
HYPOCHLORINATION STATION

Figure 23.

Dugway Brook sampling stations

Table 2. SAMPLING LOCATIONS FOR DUGWAY BROOK AFTERGROWTH PROJECT

Station	Description
1	At chlorination facility
2	200 feet downstream of chlorination facility in middle of stream
3	1300 feet downstream of chlorination facility in middle of stream
4	1800 feet downstream of chlorination facility in middle of stream
5	Mouth of stream (2200 feet downstream)
6	In the lake, 200 feet west of the mouth, 50 feet north of the shore
7	In the lake, 50 feet north of the stream mouth
8	In the lake, 100 feet north of the stream mouth
9	In the lake, 200 feet east of the stream mouth, 50 feet north of the shore
10	In the lake, 500 feet east of the stream mouth, 50 feet north of the shore
11	In the lake, 1300 feet east of the stream mouth, 50 feet north of the shore

Results of the chemical analyses are given in Appendix E for the stream and lake sampling stations. The concentration of many of the chemical parameters decreases toward the mouth of the stream. This trend corresponds with the general decrease in the levels of bacteria downstream of Station 1.

RESULTS - DURING CHLORINATION

Aftergrowth in the stream was evident in each of the three groups of indicator bacteria during the chlorination period. Chlorination initially caused a rapid decrease in bacterial levels (Station 2), but aftergrowth could be seen by the time Station 3 was reached. Each group continued to increase in absolute levels until the stream emptied into the lake.

Levels of bacteria in the lake appeared to stabilize with minor fluctuations. Figure 24 shows the effects of chlorination and aftergrowth of fecal coliforms when compared to the period prior to chlorination.

Subsequent laboratory studies indicated that peak aftergrowth occurs between 2 and 3 days after chlorination. The amount of time for flow from the stream chlorination station to the mouth was calculated to be only 16 hours. The laboratory data indicated that, by chance, the chlorination station on this stream was established at a critical point. If the station had been further upstream, the level of fecal coliforms in the lake probably would have been higher than was observed, due to a longer period for aftergrowth before dilution at the stream mouth. If the station had been further downstream, the level of fecal coliforms in the lake would probably have been below 200 per 100 ml due to a shorter period of aftergrowth prior to dilution. The residual chlorine was not totally dissipated until approximately 12 hours downstream of the hypochlorination station. The bacteria had only three to four hours to exhibit aftergrowth.

Analysis for Salmonella during the chlorination period indicated that chlorination causes an initial reduction in their numbers, as with the indicator group of bacteria.

Results from chemical analyses indicate differences between several of the parameters for the chlorinated and non-chlorinated periods. Except for proteinaceous material, no strong correlations could be seen with bacterial growth patterns before and during chlorination. Further, the differences in water chemistry cannot be assigned to chlorination effects because of the different seasonal times for the chlorinated and non-chlorinated periods.

Table 3. DETECTION OF SALMONELLA IN DUGWAY BROOK

Date	Location	Sample Size	Presence of Salmonella
		Prior to Chlorination	
4-15	St. 1	100 ml	Positive
4-15	St. 4	100 ml	Positive
5-18	St. 1	50 ml	Positive
5-18	St. 4	50 ml	Negative
6-10	St. 1	50 ml	Positive
6-10	St. 4	100 ml	Positive
		During Chlorination	
7-12	St. 1	100 ml	Positive
7-12	St. 4	500 ml	Negative
8-10	St. 1	50 ml	Positive
8-10	St. 4	500 ml	Negative
8-20	St. 1	100 ml	Positive
8-20	St. 4	1700 ml	Negative
8-25	St. 1	100 ml	Positive
8-25	St. 4	2000 ml	Positive

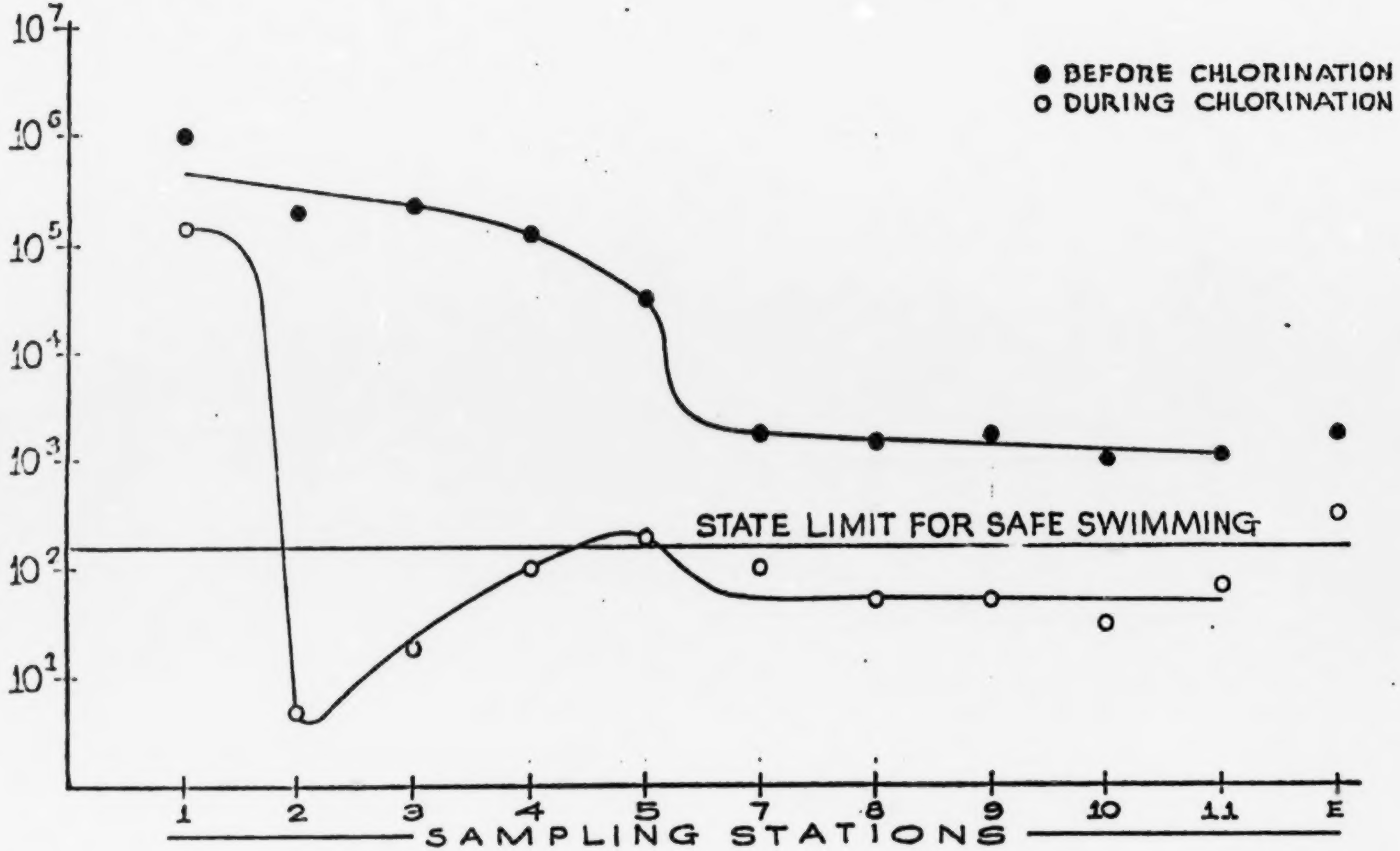


Figure 24.

The effect of in-stream chlorination (3 mg residual) and aftergrowth on lake levels of fecal coliform

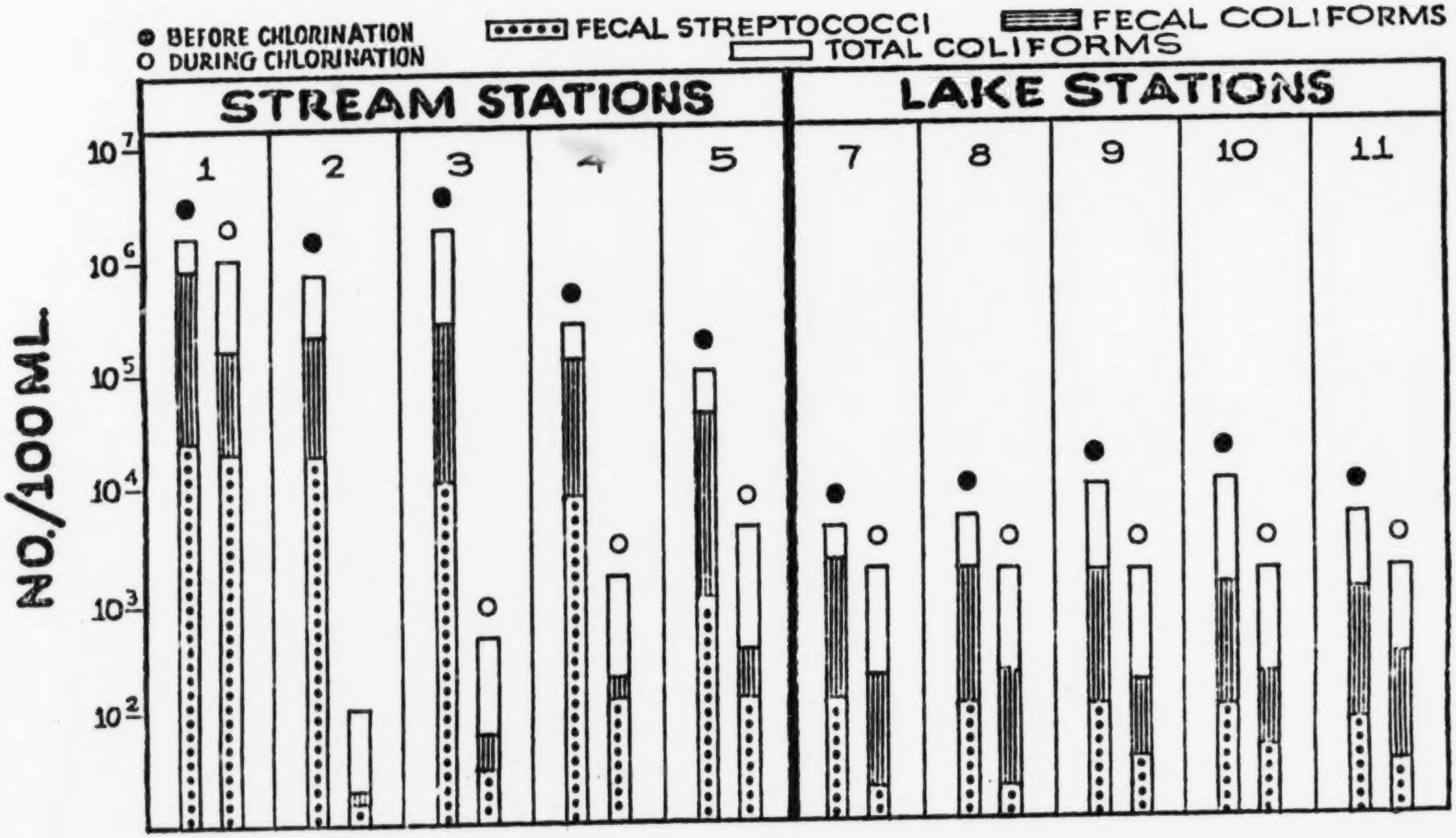


Figure 25.

Aftergrowth of total coliforms, fecal coliforms, and fecal streptococci in Dugway Brook

LABORATORY STUDIES

Several bench-scale studies were undertaken in the laboratory to gain some quantitative information on the relationship between chlorination of wastewater and subsequent bacterial aftergrowth. These laboratory studies centered on possible aftergrowth relationships.

1. Influence of particle-size distribution in wastewater of the effectiveness of chlorination and aftergrowth.
2. Effects of extended-time, chlorine contact on bacterial aftergrowth.
3. Effects of dilution on aftergrowth.
4. Effects of chlorination on proteinaceous material.

INFLUENCE OF PARTICLE SIZE

The efficiency of chlorination as a bacteriological process is partly related to the particle-size distribution in wastewater. If large aggregates are present, many bacteria will not be effected by initial chlorination, and as the particles subsequently break down, bacteria could be released to serve as a new inoculum. This phase of the laboratory studies was designed to determine the extent of the relationship existing between particle-size distribution at the time of chlorination and subsequent levels of bacterial aftergrowth as applicable to the hypochlorination of streams.

METHODS

Five ten gallon splits from a single sample of water taken from Dugway Brook each received one of the following treatments:

1. Blended in a Waring blender at high speed for 2 minutes.
2. Blended then chlorinated for 30 minutes.
3. Chlorinated for 30 minutes, neutralized, then blended.
4. Chlorinated for 30 minutes.
5. Untreated. (Control)

Thirty minute chlorination was at 3 mg/1 residual chlorine using sodium hypochlorite. After the thirty minute chlorination period, residual chlorine was neutralized with sterilized sodium thiosulphate. Care was taken to insure that the temperature of the sample did not vary by more than two degrees Centigrade.

Each of the five samples was subdivided after treatment into five sub-samples (one gallon each). Four of the sub-samples were used for chemical analyses (one gallon each) and one sub-sample was used for bacteriological analyses (one-half gallon). All sub-samples were incubated at 25°C.

Chemical analyses were run initially at 24 hour intervals for three days. Following treatment, bacteriological analyses were run at 3, 6, 9, 18 and 24 hour intervals during the first day, and thereafter at 12 hour intervals up to 96 hours post treatment. Levels of total bacterial, total coliform, fecal coliform, and fecal streptococcus were measured.

Particle-size distribution were calculated from measurements made with a Whipple ocular micrometer. For each measurement, 25 ml of sample were withdrawn and passed through a clean membrane filter. The filter was dried in a dessicator and the dried filters were covered with immersion oil to make them transparent prior to reading under the microscope.

RESULTS

Figures 26-27 summarize results for the effects of the various treatments on bacterial growth and particle size distribution. A comparison of the data between the control sample and the blended sample shows that a reduction in particle size did not appreciably affect bacterial growth. The rates of bacterial decline were similar between the two treatments, although blending appeared to slightly lessen the rate of decline, and the levels of bacteria at the end of 72 hours were not significantly different.

There was an apparent difference between the samples that were chlorinated and samples that were blended then chlorinated. Although the peak levels of aftergrowth did not differ, the blending-then-chlorinating treatment resulted in a faster rate of aftergrowth than did chlorination alone.

Among the four groups of bacteria for which analyses were made, the fecal streptococcus group exhibited a very limited ability for aftergrowth. Fecal coliforms displayed a moderate ability for aftergrowth, but their peak aftergrowth levels did not exceed the initial level of bacteria. Both the total bacteria and total coliform groups were capable of aftergrowth that closely approximated, or exceeded, their respective

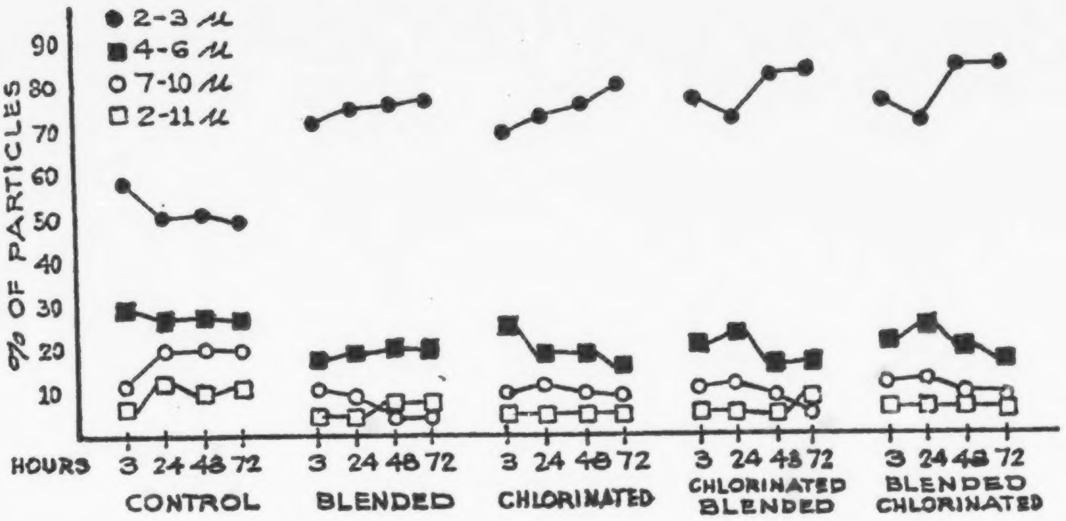


Figure 26.

Treatment influence on particle size

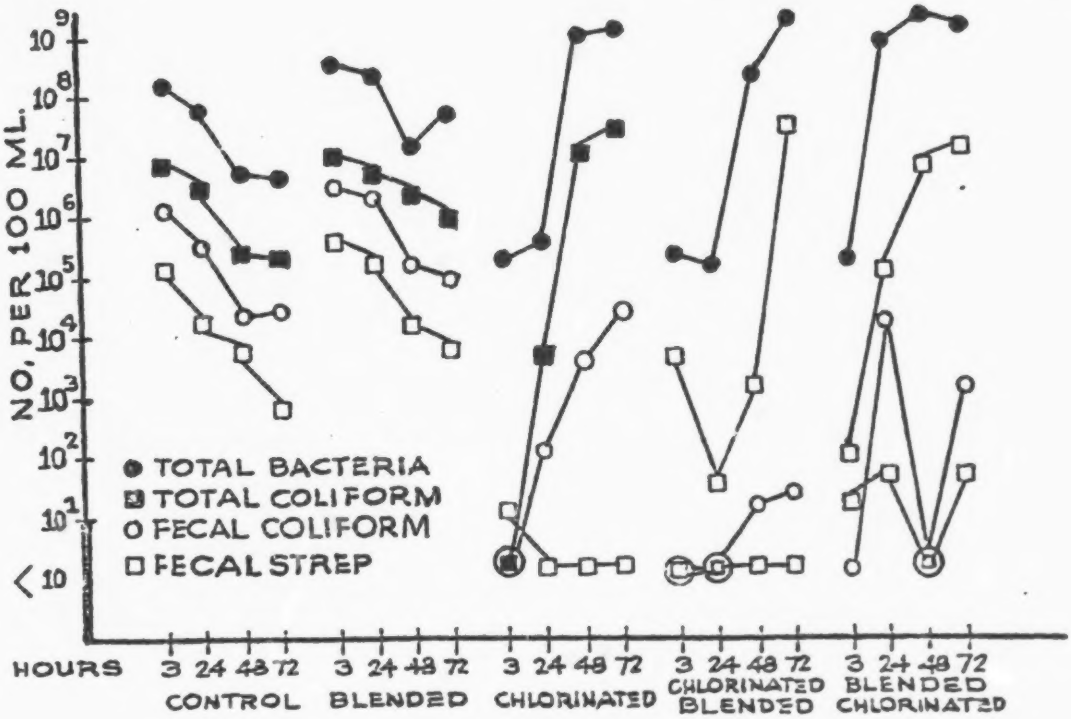


Figure 27.

Treatment influence on bacteria

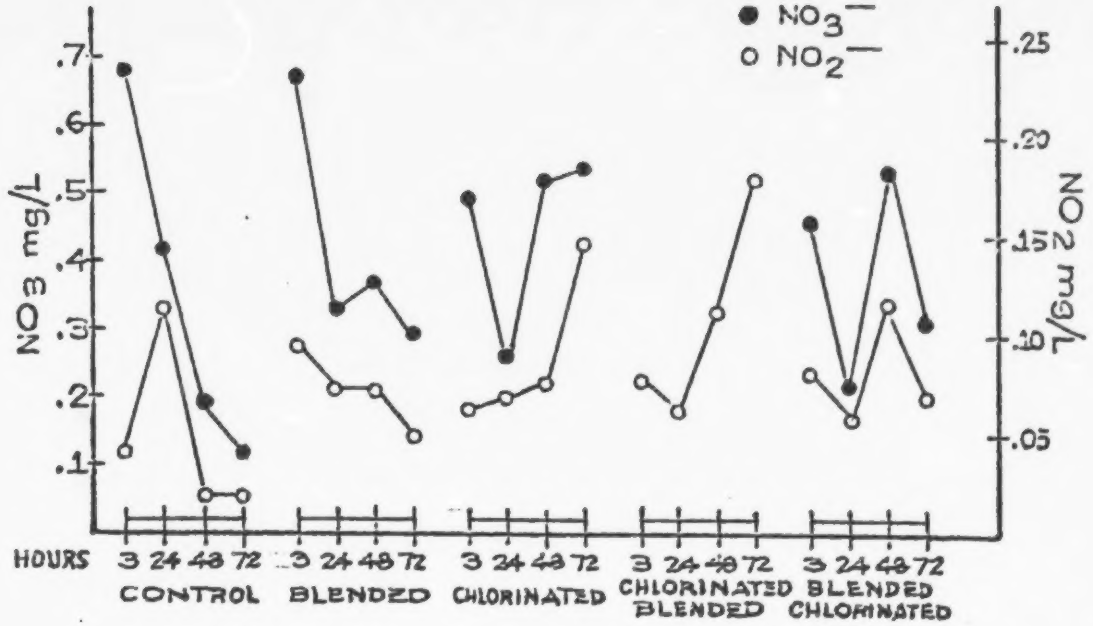


Figure 28.

Treatment influence on
Nitrite and Nitrate

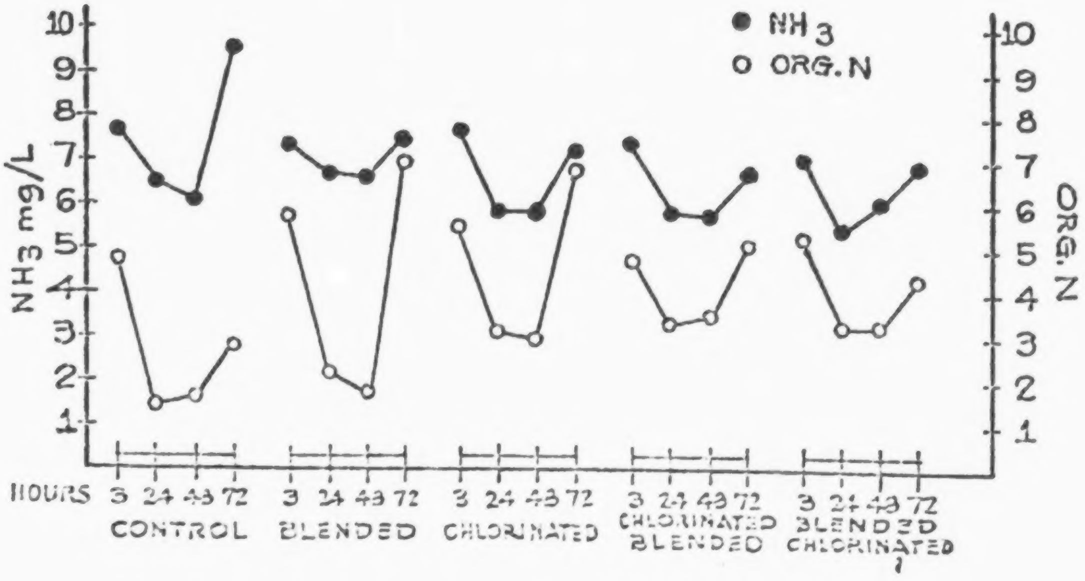


Figure 29.

Treatment influence on
Ammonia and Organic Nitrogen

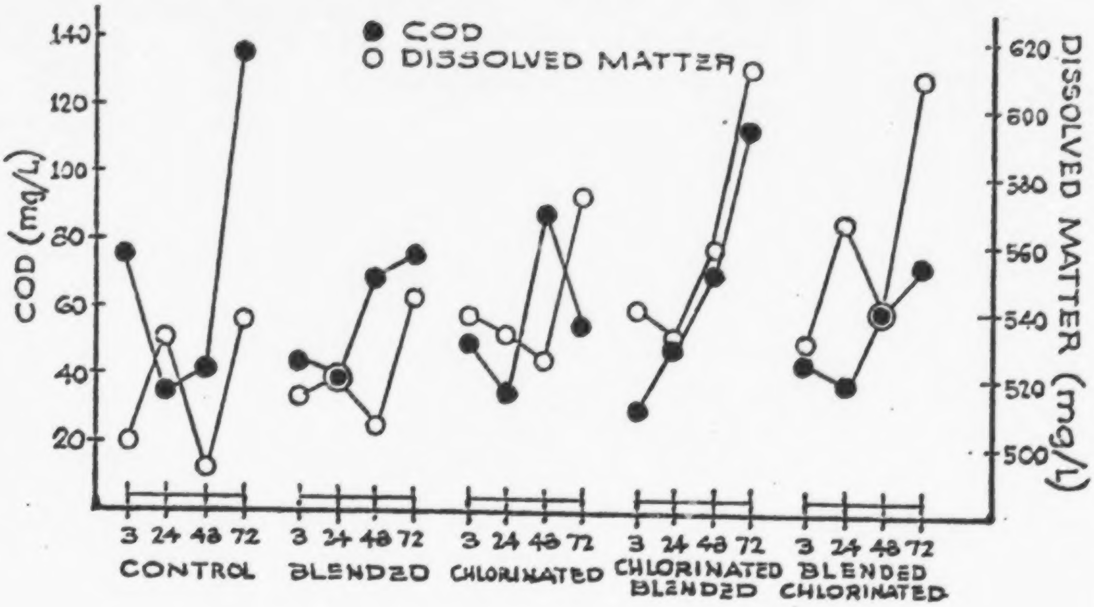


Figure 30.

Treatment influence on
COD and Dissolved Matter

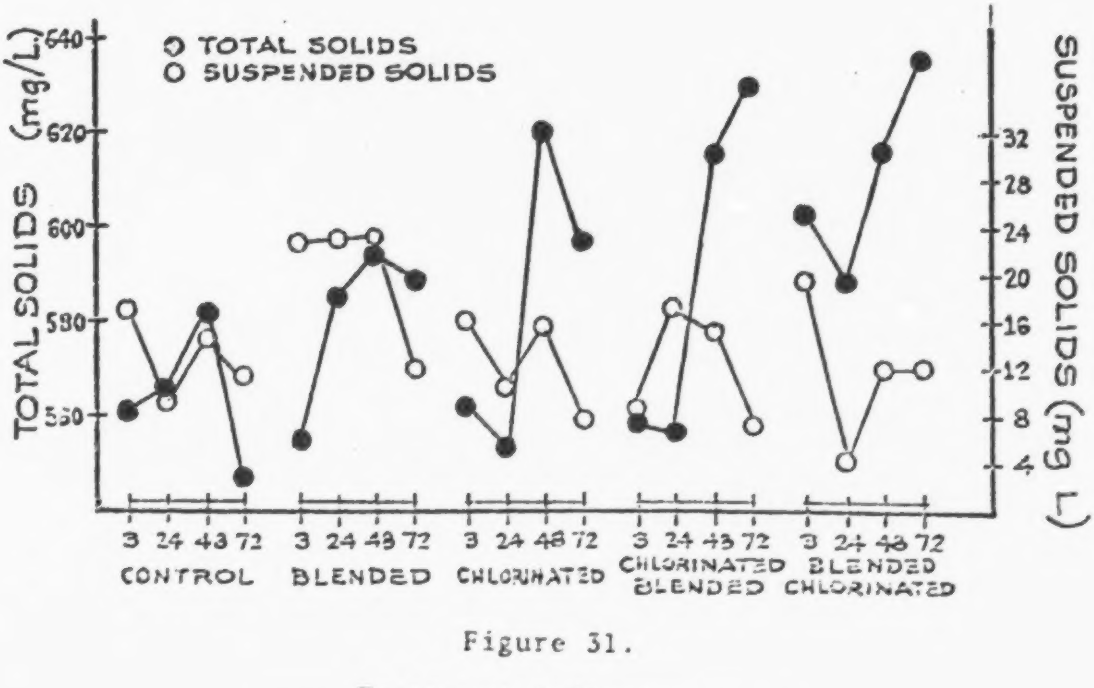


Figure 31.

Treatment influence on
Total Solids and Suspended Solids

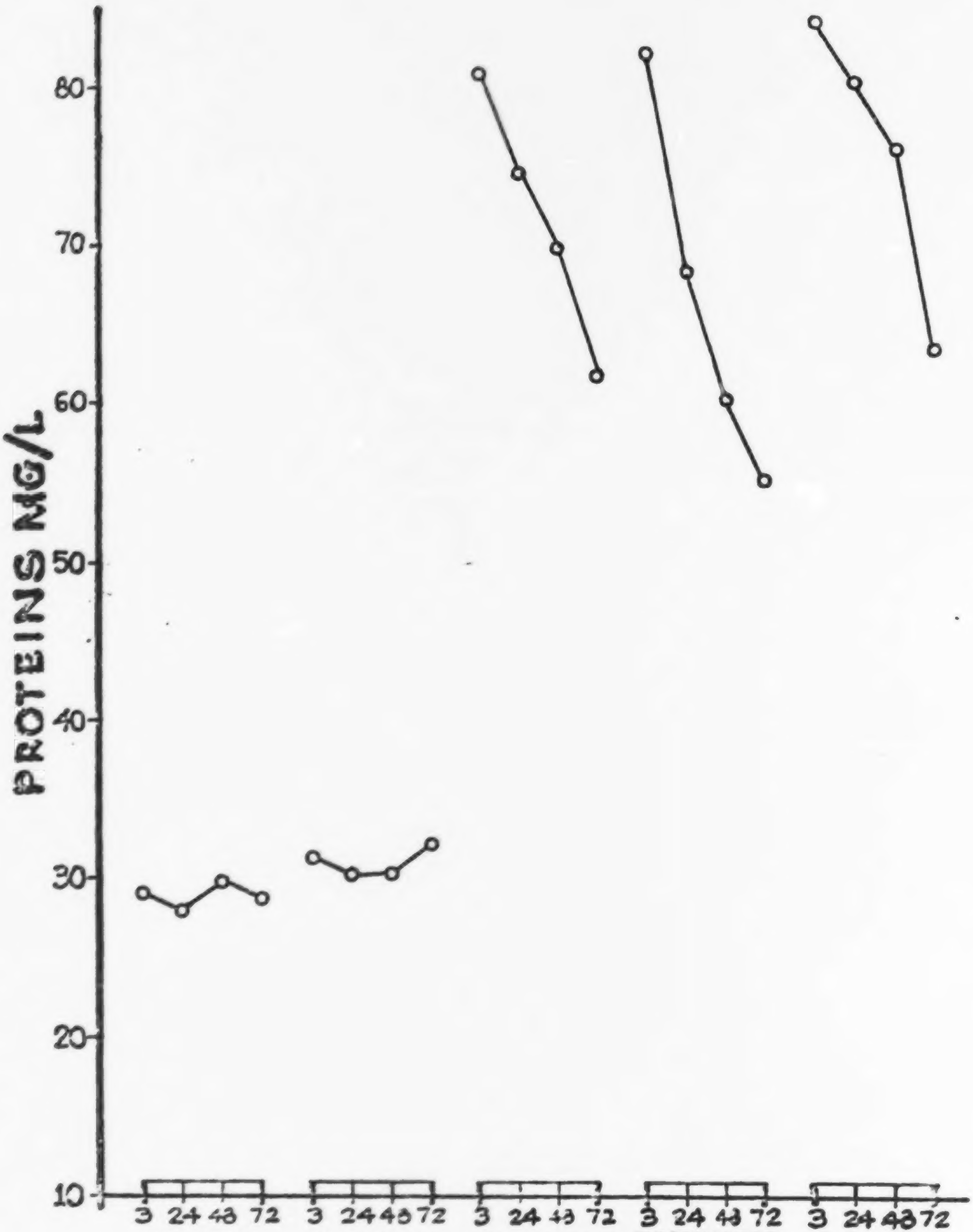


Figure 32.

Treatment influence on Proteins

initial levels. Typical chemical results are shown in Figures 28-32. Complete results from the chemical analyses are given in Appendix E. The only significant chlorine-induced changes in the chemical environment of the stream water was with proteinaceous material. Chlorination appeared to have increased the number of available proteins (Figure 32) by cleaving complex proteins thus making them more available for bacteria metabolism. Protein analysis was done by the Lowry Method using gelatin as the protein standard. The Lowry Method and subsequent laboratory investigations are described in Appendix E.

EXTENDED-TIME CONTACT BASIN

The purpose of this study was to determine if: (1) the length of contact time in chlorination was related in some way to bacterial aftergrowth, and (2) dilution of the chlorinated water had a significant affect on aftergrowth. Initially, it was felt that several possible relationships could exist, some resulting in decreased or no aftergrowth and some resulting in increased aftergrowth.

The length of chlorine contact could affect aftergrowth through two major factors: (1) nutrients for bacteria, and (2) level of bacteriological action. For example, if extended chlorine contact (72 hours) with wastewater should increase the initial bacterial kill as compared with short term contact (15 minutes) but increase available nutrients for bacteria that remain, undesirable aftergrowth may occur. Conversely, extended contact time may not increase bacterial kill, but may reduce available nutrients, resulting in decreased or non-existent aftergrowth. Dilution after chlorination could either enhance or reduce contact-time effects.

METHODS

Two 55 gallon aquaria, each containing 50 gallons of water from Dugway Brook, were set up in the laboratory. One tank was left untreated for a control. The second tank was treated with sodium hypochlorite to produce a residual chlorine of 6 mg/l. Both tanks were continually agitated during the entire experiment.

After 15 minutes of contact time, a sample was taken from the treated tank, and its residual chlorine was neutralized with sodium thiosulfate. The neutralized sample was split and partially diluted with lake water. Chlorinated-then-dechlorinated samples of 100%, 75%, 50%, 25% and 10% Dugway Brook water were obtained in this manner.

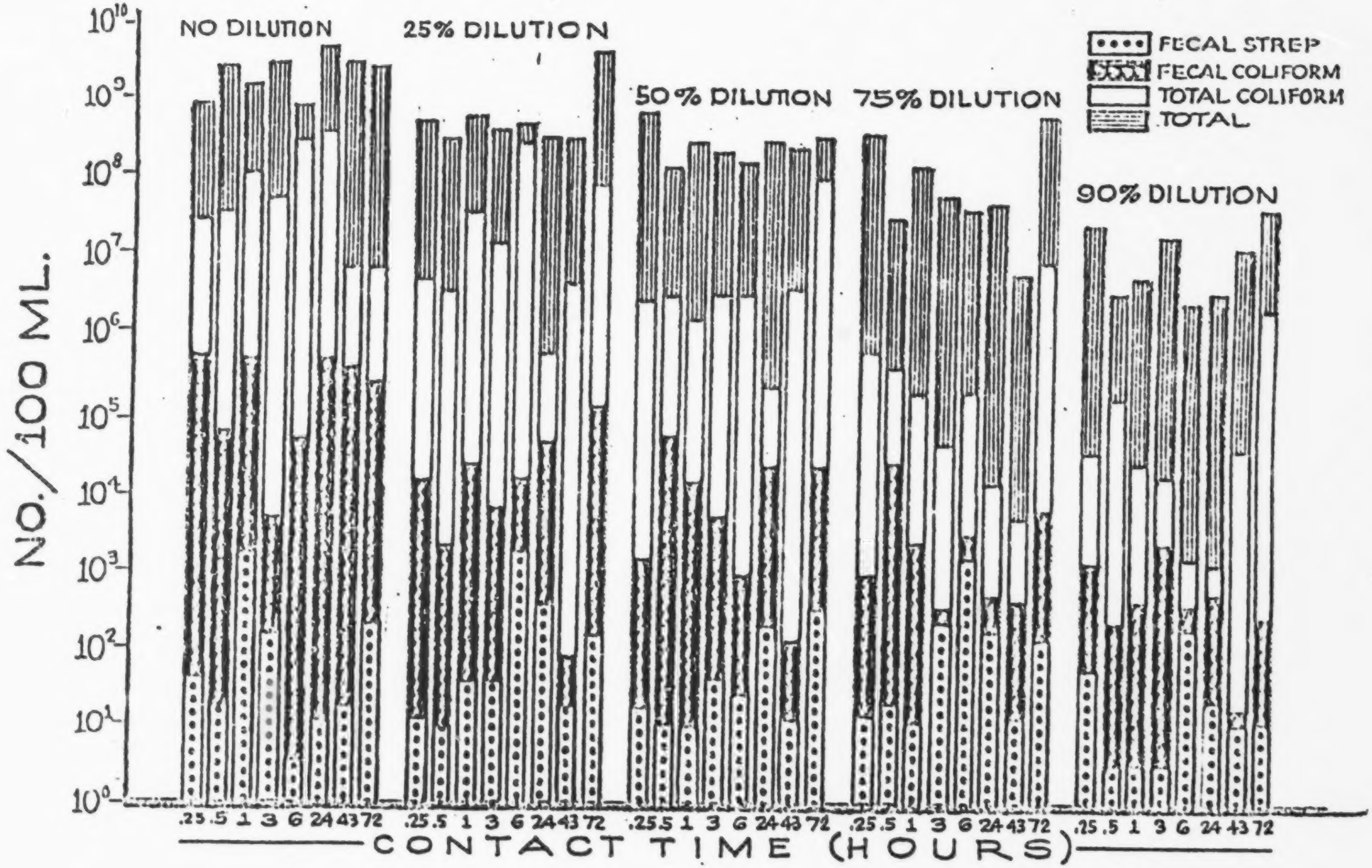


Figure 33.

Effect of chlorination contact time and dilution on peak levels of aftergrowth in Dugway Brook water

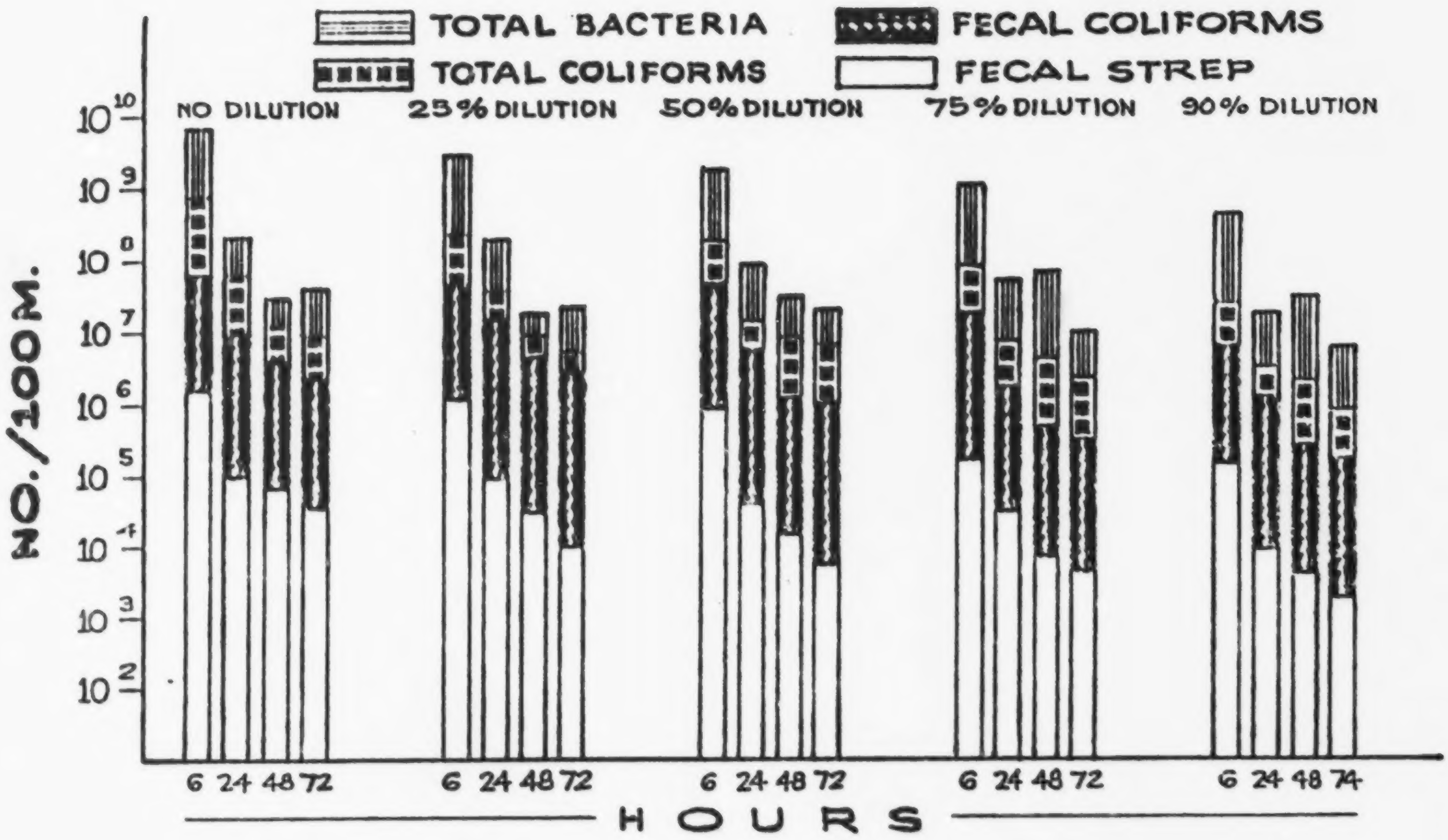


Figure 34.

Effect of dilution on peak levels of bacteria
in Dugway Brook water

These samples were analyzed for total bacteria, total coliforms, fecal coliforms and fecal streptococcus immediately after dilution, and at 24, 48, and 72 hours.

This procedure for the 15 minute contact time was repeated for contact times of 30 minutes, 1 hour, 3 hours, 6 hours, 24 hours, 48 hours, and 72 hours. A residual chlorine level of 6 mg/l was maintained throughout the full 72 hour contact time. Samples were withdrawn from the control tank on the same time schedule as for the treated tank, diluted in an identical manner, and analyzed for bacteria as described above.

Chemical analyses were performed on samples taken from both the control and treated tanks; initially, at 24, and 72 hours.

RESULTS

Immediately after chlorination, bacterial levels were greatly reduced for all contact times. Peak levels of aftergrowth following dechlorination have been plotted in Figures 33-34. The length of chlorine contact did not significantly affect the aftergrowth level for any of the four groups of bacteria that were measured. There was a large amount of variation within this data, but for the four groups of bacteria, peak aftergrowth levels following 15 minute contact time did not significantly differ from aftergrowth levels following longer contact times.

The greatest impact on the resultant levels of aftergrowth in this study resulted from dilution following chlorination-dechlorination. Peak aftergrowth levels for total bacteria, total coliform and fecal coliform were significantly reduced by initial dilution. Peak aftergrowth for fecal streptococci appeared not to be affected by dilution.

Peak aftergrowth levels for the four groups of bacteria from the control tank were similarly affected by dilution. The peak levels varied directly with the dilution ratios.

Results of the chemical analysis are given in Figure 35. Differences in the parameters measured between the control and chlorinated tanks did not appear to be significant. Changes in the relative levels of nitrate and nitrite did occur after 24 hours, which probably reflected both the oxidative action of the chlorine in the experimental tank and oxygen depletion in the control tank. No correlation could be seen between changes in chemical parameters and

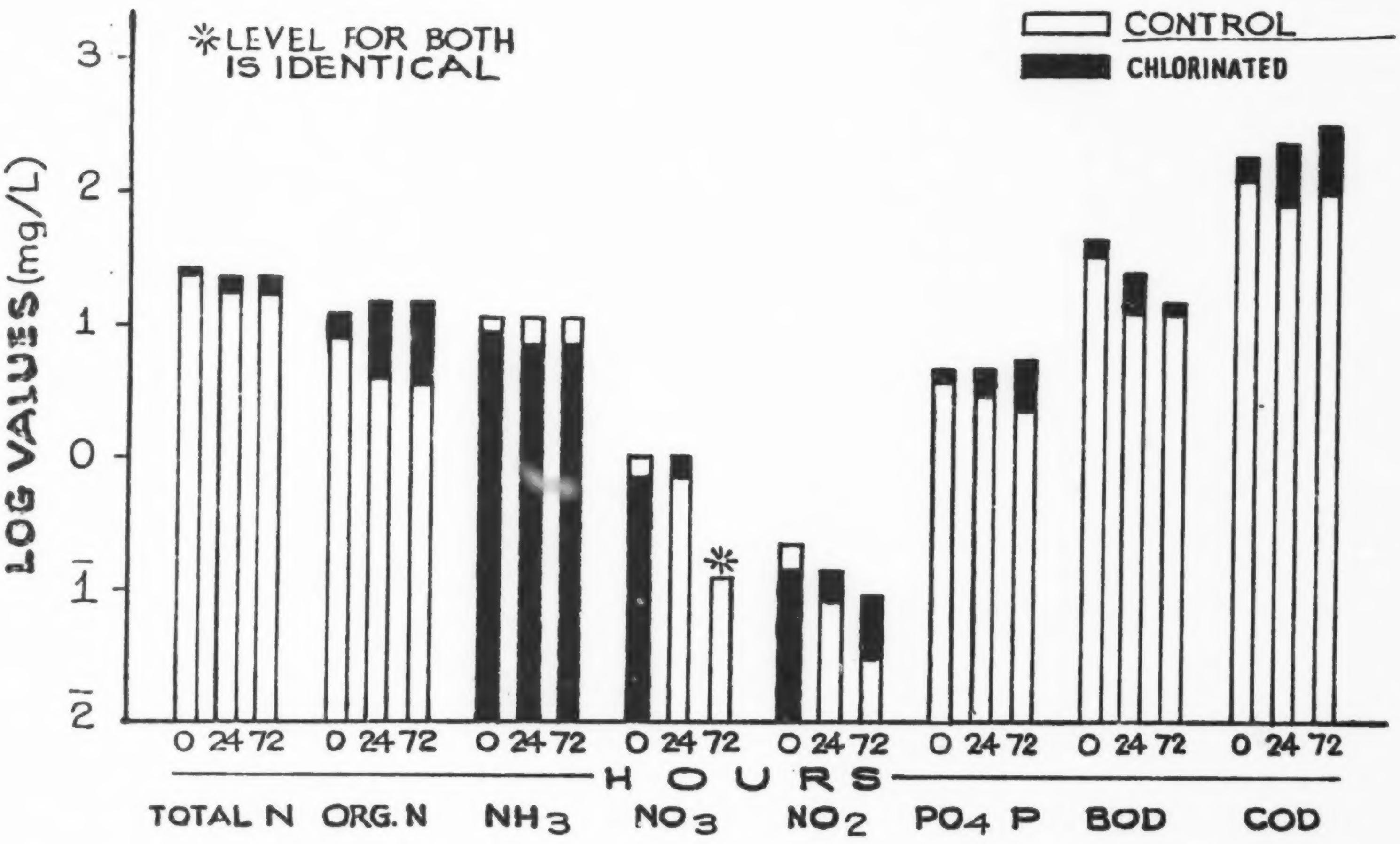


Figure 35.

Effect of chlorination contact time on selected chemical parameters patterns of bacterial aftergrowth in either the control or chlorinated tank.

The average levels of aftergrowth for total bacteria in each dilution equalled or very nearly approached the "initial" level (6 hour level) of total bacteria in the untreated samples. Because the number of total bacteria decreased with time in the untreated samples, and the peak level of total bacteria in the chlorinated-dechlorinated samples increased with time to the same peak level within each dilution treatment, there were more total bacteria at the peak growth in the chlorinated-dechlorinated samples than in the untreated samples after the first 6 hours. Figure 35 shows this relationship for undiluted and 10% Dugway water.

Fecal coliforms did not exhibit the same potential for aftergrowth as total bacteria, but aftergrowth was still evident. It is of significance to note that, while chlorination alone did not appreciably affect the level of this group of bacteria when compared to the control group, chlorination-with-dilution resulted in a significant decrease when compared to the diluted control.

It should also be noted for fecal coliforms that, while chlorination and dilution of the water to 10% of its original concentration did significantly reduce the final level of bacteria, the level of aftergrowth exceeded 200 fecal coliforms per 100 ml. This figure is the basis for maximum allowable levels of fecal coliforms in body contact recreation waters of Ohio.

Fecal streptococcus showed aftergrowth, but the magnitude was far less than was evidenced for fecal and total coliforms. Figure 33 shows the degree of aftergrowth for fecal streptococci.

DISCUSSION

The disinfectant quality of the sodium hypochlorite used in these studies was good, both in the field and in the laboratory. Initial kills were always sufficient to lower the levels of indicator bacteria to meet current Ohio water quality criteria for swimming. However, the occurrence of aftergrowth must be taken into account or initial chlorination benefits can be negated.

Except for the increase in reactive proteinaceous material, chemical analysis failed to show appreciable affects of chlorination on water chemistry in terms of the quality of the water for bacterial growth. For example, nitrates, nitrites, and particle size distribution were affected by chlorination, but the observed changes in these parameters were not consistent with bacterial growth patterns. From our data, it appeared that bacterial aftergrowth was not caused by chlorination induced changes in the chemical environment.

These studies point out an important difference between chlorination treatment per se and what can be termed "functional chlorination treatment". Functional chlorination describes the ultimate reduction of pathogenic organisms to safe levels while chlorination per se may describe only an initial reduction of these organisms. Aftergrowth is the variable that can cause chlorination to be non-functional.

The ultimate gain from an in-stream chlorination treatment procedure is highly dependent on: (1) the distance of the chlorination facility from the stream mouth as it affects time available for aftergrowth; and (2) the degree of dilution at the mouth. These two factors must be taken into account if a functional chlorination treatment program is to be developed.

Further studies will have to be undertaken in order to more clearly define the quantitative aspects of the above relationships, but for the conditions met in our studies it appeared that functional chlorination would result from dilutions larger than ten to one (10 to 1) of the chlorinated stream water, if the dilution occurs within 20 hours of chlorination. Dilutions less than ten to one in the laboratory resulted in aftergrowth of fecal coliforms exceeding the Ohio State criteria for swimming of 200 per 100 ml of water. In the field, the dilution was greater than 10 to 1, but it was found that if enough time was available for aftergrowth, the dilution potential for reducing fecal coliforms below 200 per 100 ml, could be negated. This has occurred particularly in Shaw Brook and Nine Mile Creek hypochlorination studies (Section V).

The above times and solution ratios most certainly do not have broad application to all areas. Local conditions must be considered such as the pre-chlorination levels of bacteria and the quality of the dilution water. However, these factors should be considered in relation to time available for aftergrowth and degree of dilution.

SECTION VII

OVERFLOW REDUCTION BY SEWER FLUSHING

INTRODUCTION

In an attempt to reduce the level of pollutants reaching the shores of Lake Erie, several methods of reducing pollution at its source have been investigated. Among the various methods investigated was the flushing of combined sewers in residential areas. These lateral sewers often sustain a low flow during dry weather. The low flow allows the accumulation of high oxygen demanding solid waste. In periods of high sewer flow, as during rainstorms, accumulated mass is washed out of the sewer. Large quantities of this mass escape sewage treatment via overflow because combined sewers in Cleveland are unable to handle the full burden of the storm water. Regular flushing of the lateral sewers during dry weather could significantly reduce the accumulation of settled material.

Experiments were conducted to determine both the magnitude of pollutants accumulated in lateral sewers and the effectiveness of three different approaches to sewer flushing. The three approaches to sewer flushing were:

Phase I	Introduction of water directly into manholes
Phase II	Filling of manholes followed by the instantaneous release of the water
Phase III	Street washing with the resulting flow entering a catch basin, then into the sewer.

TEST AREA

The area enclosed by West 102nd, West 103rd Streets, Baltic Road, and Clifton Boulevard was selected for the sewer flushing tests (see Figure 36). The study area is located in a typical residential area in Cleveland with combined sewers and a drainage area serving a relatively small number of homes. The size of the two sewers in this area is 12 inches and the slope is less than 1%. It was advantageous to run the flushing test on a sewer with a gentle slope because accumulation of solids would be more likely to occur. If the tests were successful in these sewers the techniques used should also be useful in areas of Cleveland having sewers with greater slope.

METHODS TESTED

Phase I

After background work was performed to calculate fire hydrant discharge rates, expected ranges of pollutants and velocities (Appendix F), Phase I of the Sewer Flushing Program was ready to begin.

Using a firehose, water from a fire hydrant was injected directly into a manhole to produce a continuous rate of flow. A moderate rate of flow, two feet per second, for 10 minutes was used to flush the sewer. After this flush period the water flow was increased to 3.7 fps for 10 minutes in order to determine the flushing efficiency at the maximum flow obtainable.

Samples and flow measurements were taken at two locations downstream. Samples were analyzed for their Chemical Oxygen Demand (COD) and suspended solids content.

Phase I Results

The suspended solids and COD data from Phase I indicated that there was an immediate release of substantial amounts of pollutants followed by a sharp reduction of this material (Figures 37-40), usually occurring after the second sample. After the second sample, the COD and suspended solids content generally decreased and then remained at a low level.

The relative amount of solids flushed out at a flow velocity of 3.7 fps was small compared to the initial flush with a flow velocity of 2 fps. This indicated that flows greater than 2 fps would not be necessary for adequate flushing of sewers.

Phase II

Phase II was designed to determine whether one large surge of water was more effective in scouring combined sewers and removing the pollutants than a lesser, regulated flow over a longer period of time.

The combined sewers were blocked with a "balloon-type" stopper where the sewers were connected to manholes A and D, the uppermost manholes on West 102nd and West 103rd Streets, respectively. The balloon-type stopper is so named because

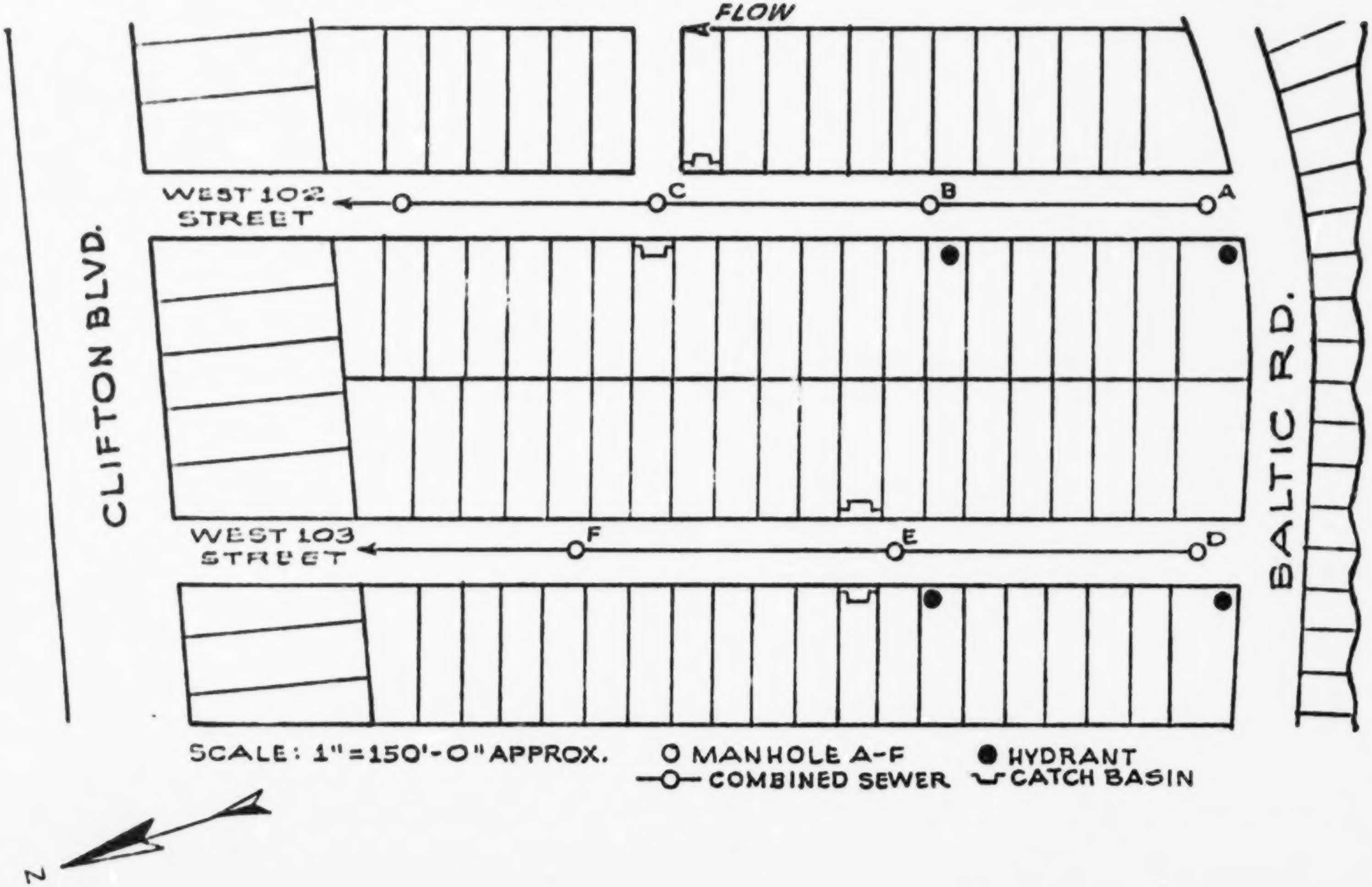

FLOW
A
B
C
D
E
F
WEST 102 STREET
WEST 103 STREET
CLIFTON BLVD.
BALTIC RD.
SCALE: 1" = 150'-0" APPROX.
○ MANHOLE A-F
─○─ COMBINED SEWER
● HYDRANT
CATCH BASIN
N

Figure 36.

Sewer flushing program study area

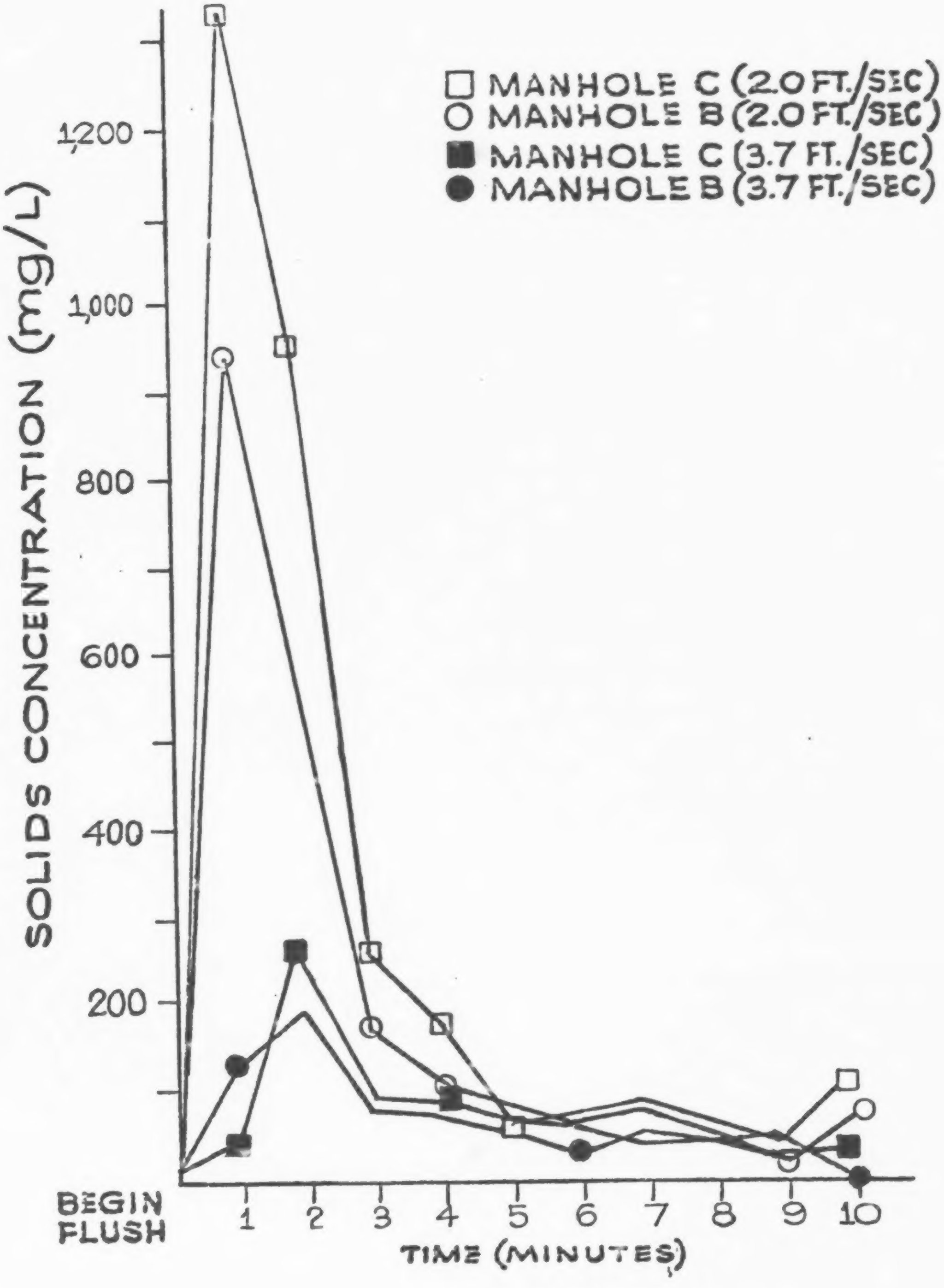


Figure 37.

Phase I - Suspended Solids
West 102nd Street

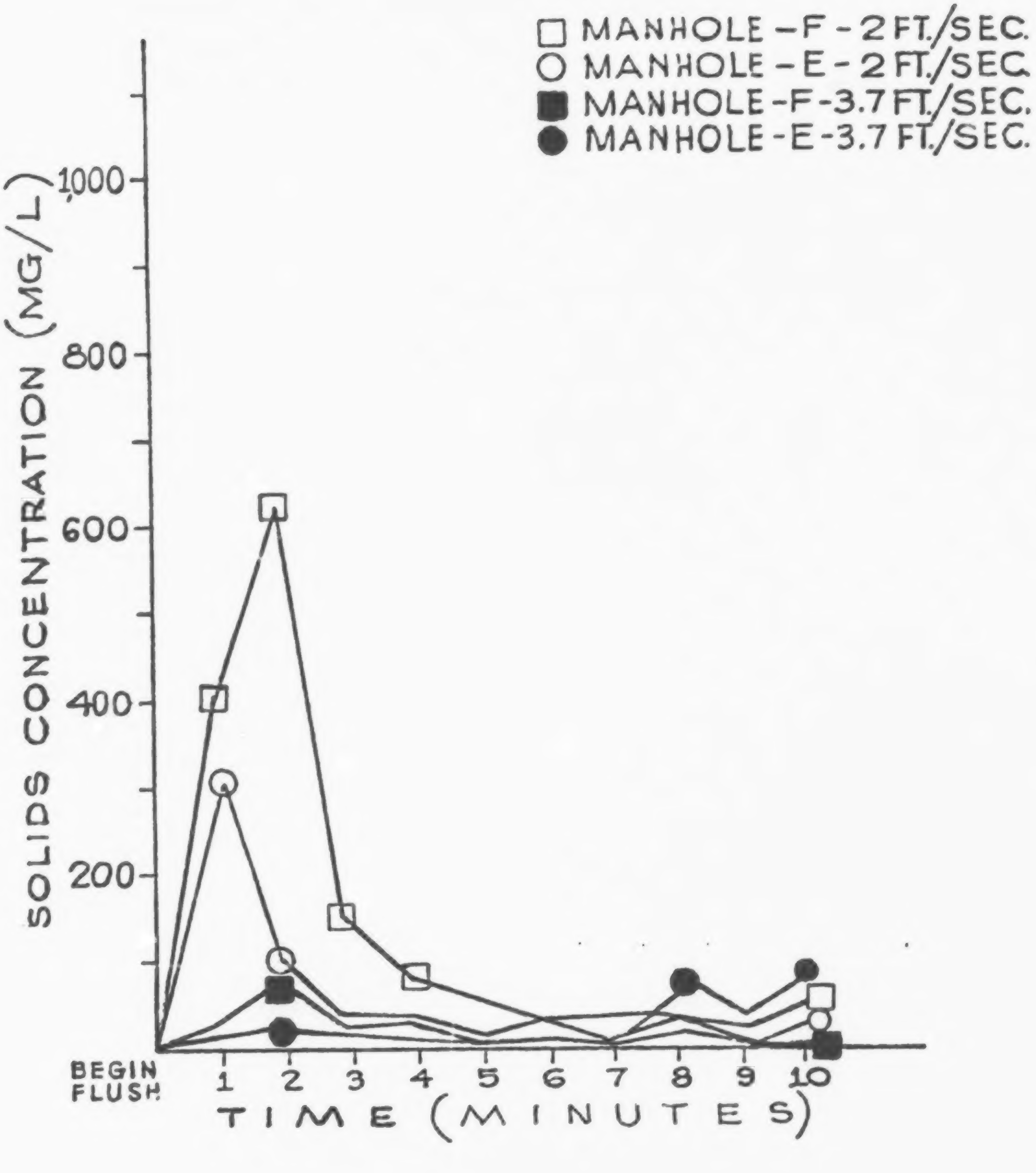


Figure 38.

Phase I - Suspended Solids
West 103rd Street

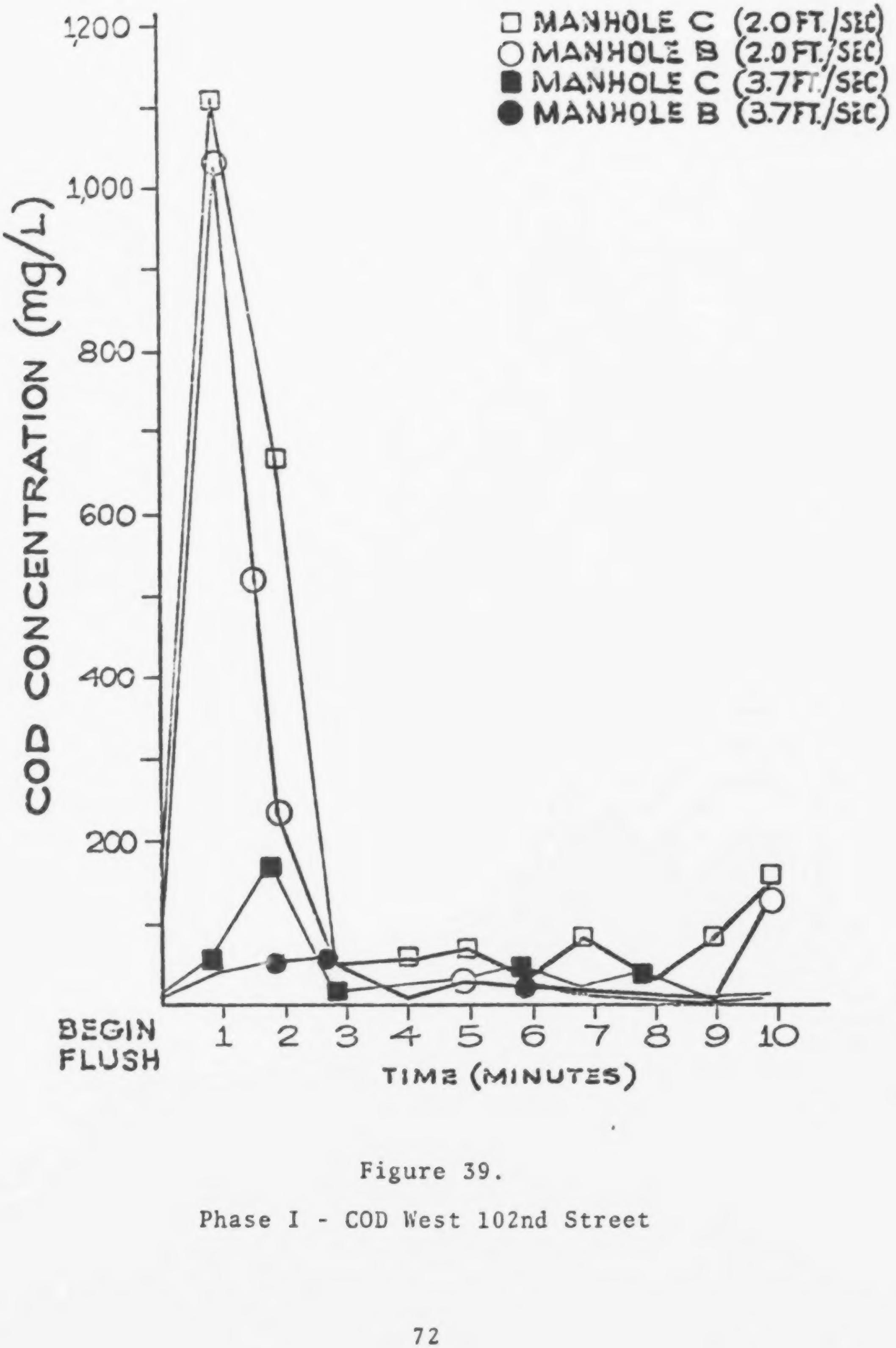


Figure 39.

Phase I - COD West 102nd Street

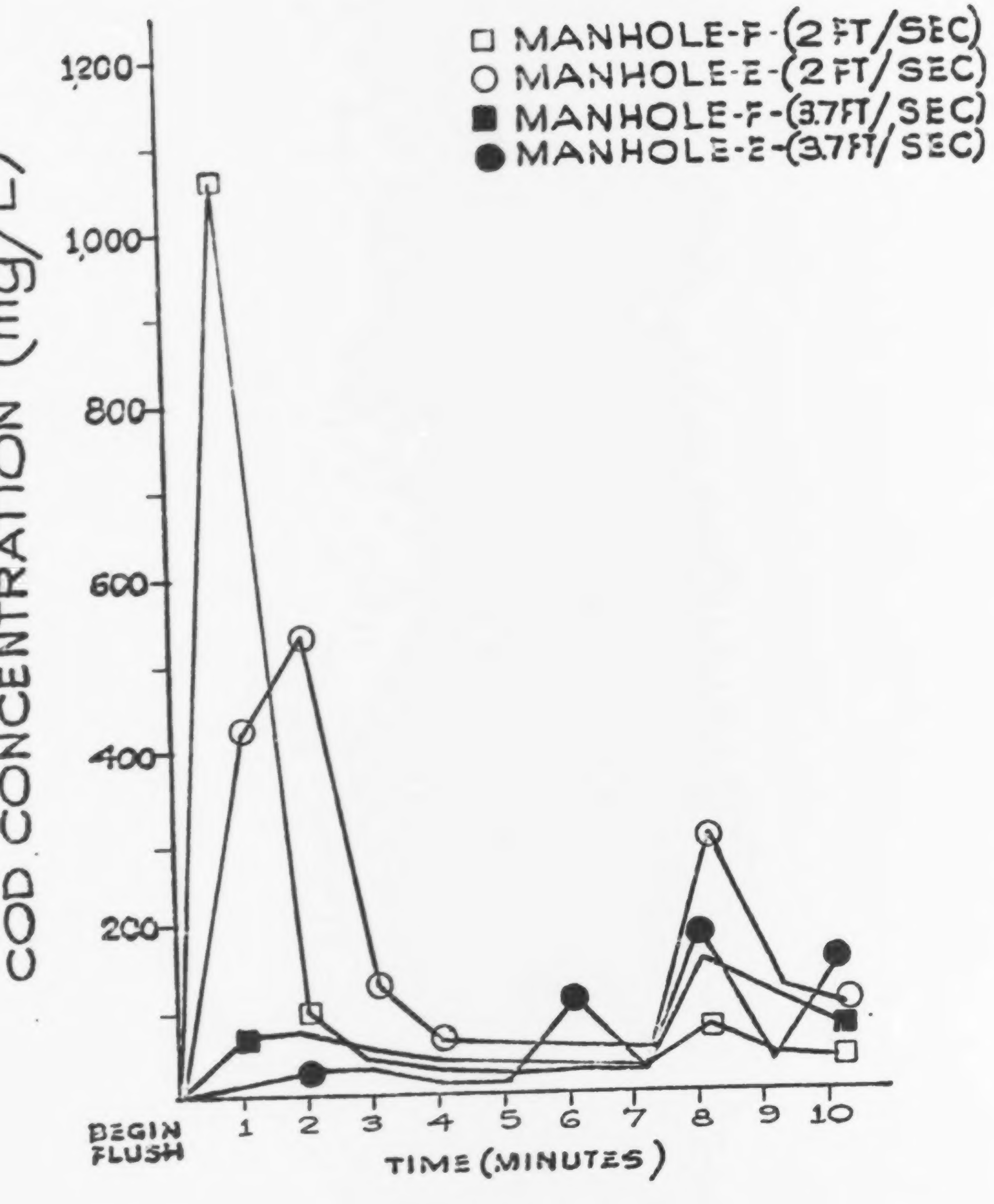


Figure 40.

Phase I - COD West 103rd Street

it is inflatable. First, the stopper was placed into the proper position with about half of the balloon inserted into the sewer. It was then inflated with a bicycle pump. The balloon was connected to a plastic tube with a valve through which the air could be released at the desired time, instantaneously releasing the water.

The manhole was filled with water from the street hydrants to a depth of seven feet. Manhole A on West 102nd Street is 8.1 feet deep, and manhole D on West 103rd Street is 8.3 feet deep. As in Phase I, two men were located at each manhole, and one man regulated the hydrant. Samples were taken every 30 seconds for five minutes after the crest passed the sampling manhole. The samples were analyzed for suspended solids and COD.

Phase II Results

There was a difference between the results from the two streets in Phase II, although the test procedures on both streets were identical (Figures 41-44). The suspended solids data on West 103rd Street appeared to follow the same pattern as in Phase I. After the sixth sample, the suspended solids content of the water increased dramatically, then returned to a low level in the final sample. The increase was especially demonstrable at manhole E where the peak reached at the eighth sample (four minutes) nearly duplicated the peak of the first sample. This increase could be attributed to flushing water backing up into sewers connecting to the houses. After the initial surge, water that had been backed up into the connecting sewers, containing large amounts of pollutant, would be released at a slower flow rate back into the combined sewer causing the second peaks after four minutes. The flushing pattern in West 102nd Street was similar to the pattern observed in Phase I, that is, a large initial increase in the suspended solids and COD followed by a sharp decline in subsequent samples. The amount of material flushed from both sewers during Phase II was less than the amount flushed in Phase I.

Phase III

Phase III of the sewer flushing program was undertaken to determine the magnitude of pollutants from street runoff in the test area, and whether or not street-washing could be combined effectively with a program of sewer-flushing. Phase III was conducted only on West 103rd Street because no catch basins existed within close proximity of a fire hydrant.

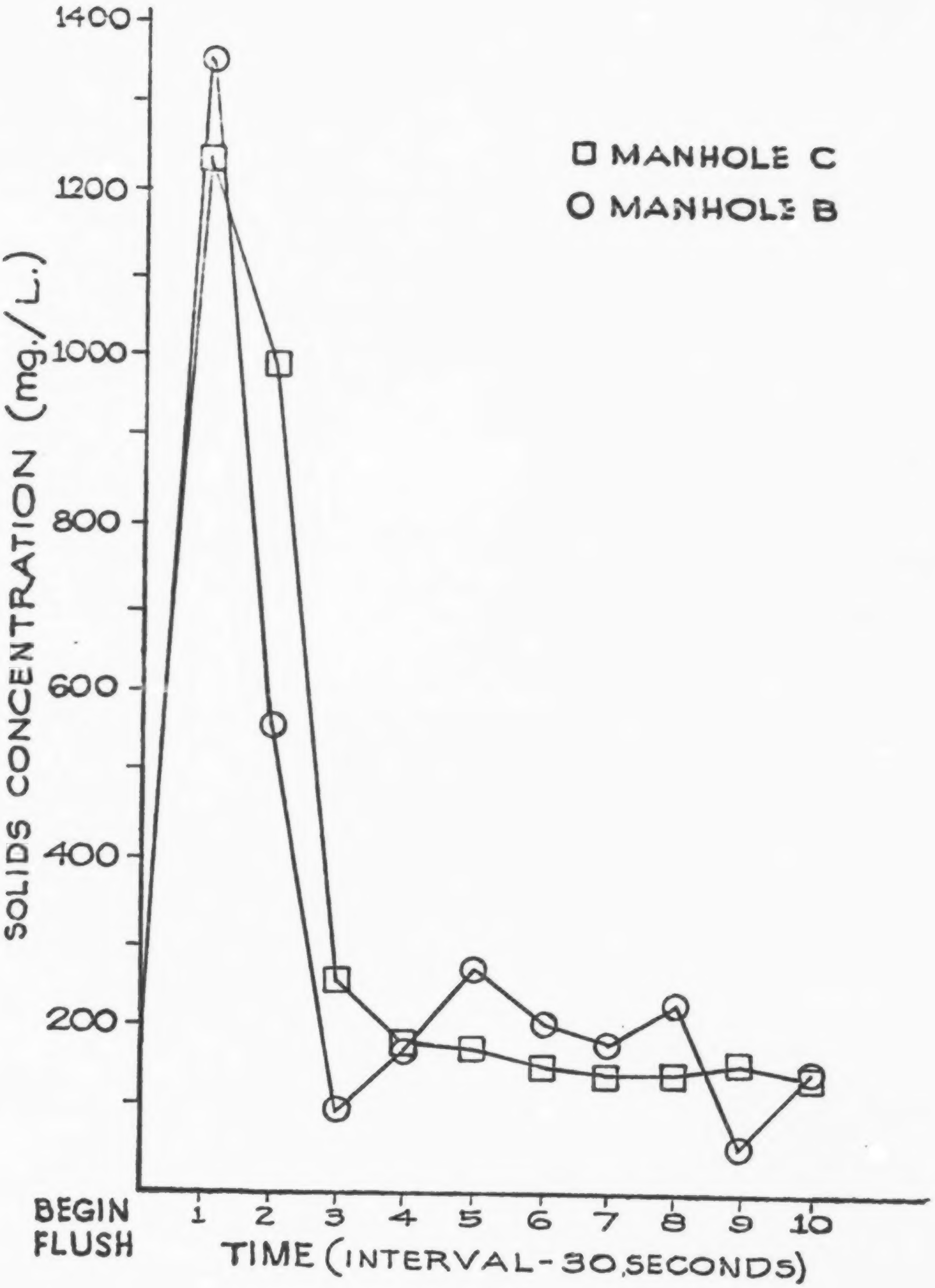


Figure 41.

Phase II - Suspended Solids
West 102nd Street

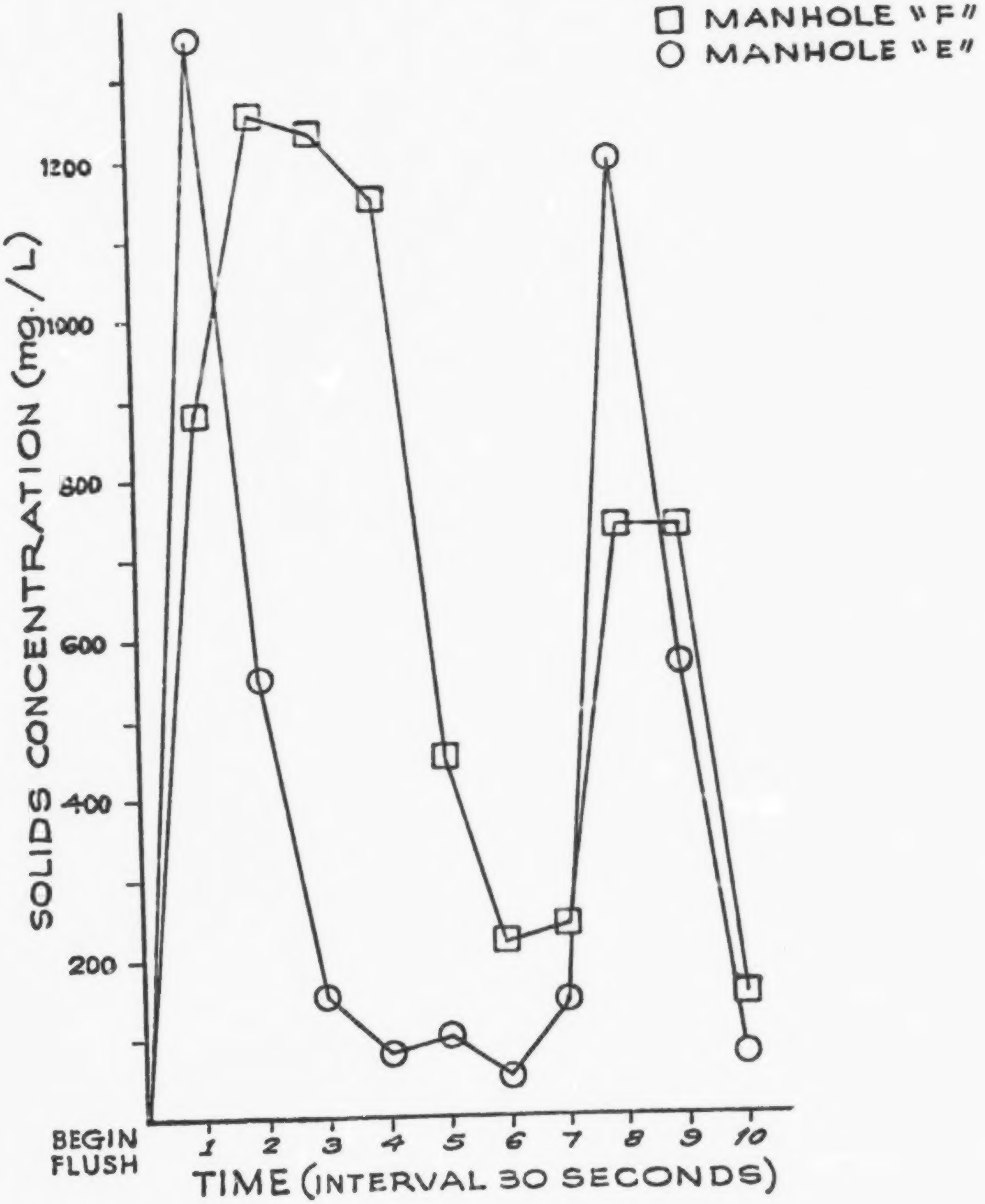


Figure 42.

Phase II - Suspended Solids
West 103rd Street

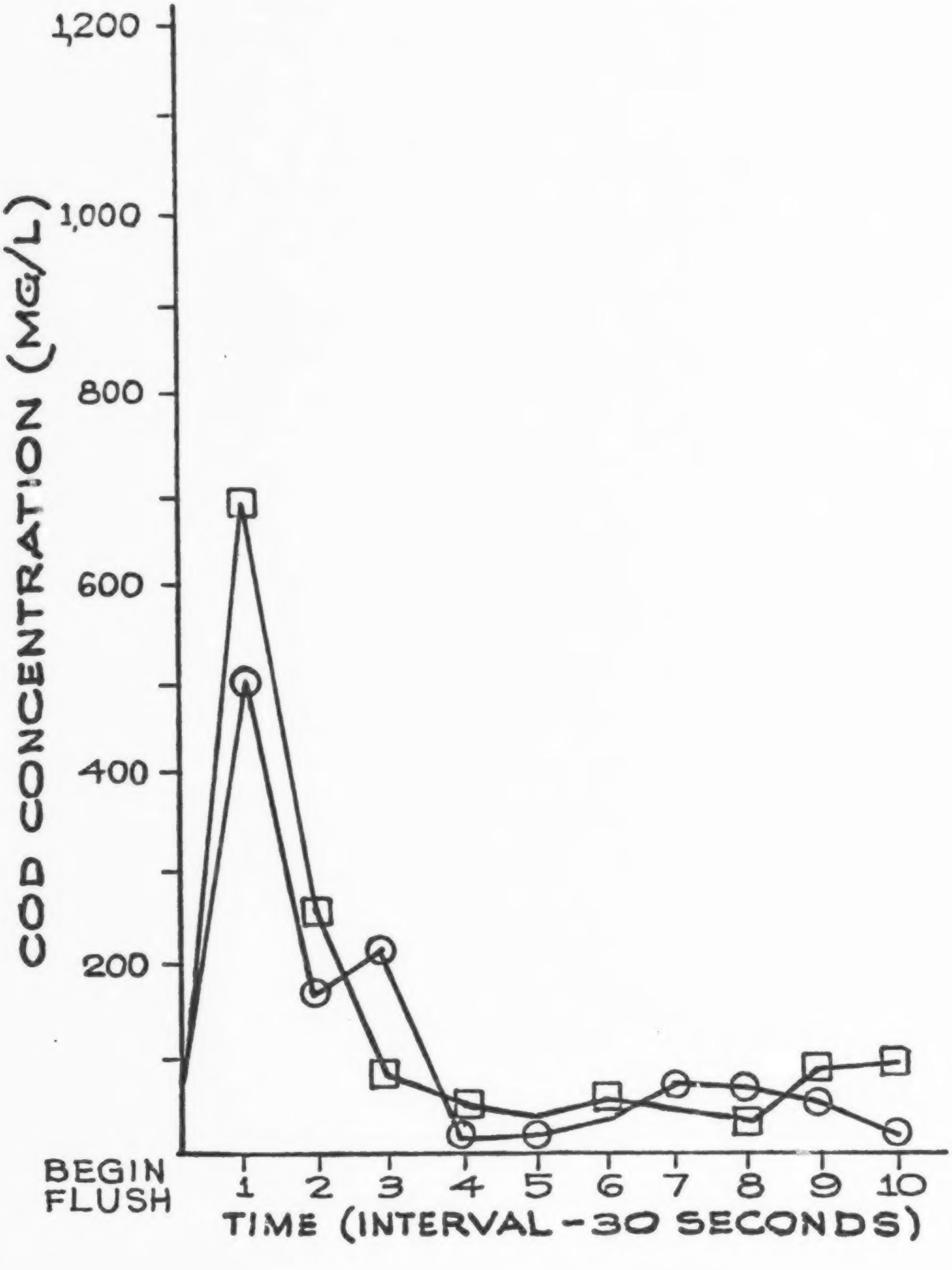


Figure 43.

Phase II - COD West 102nd Street

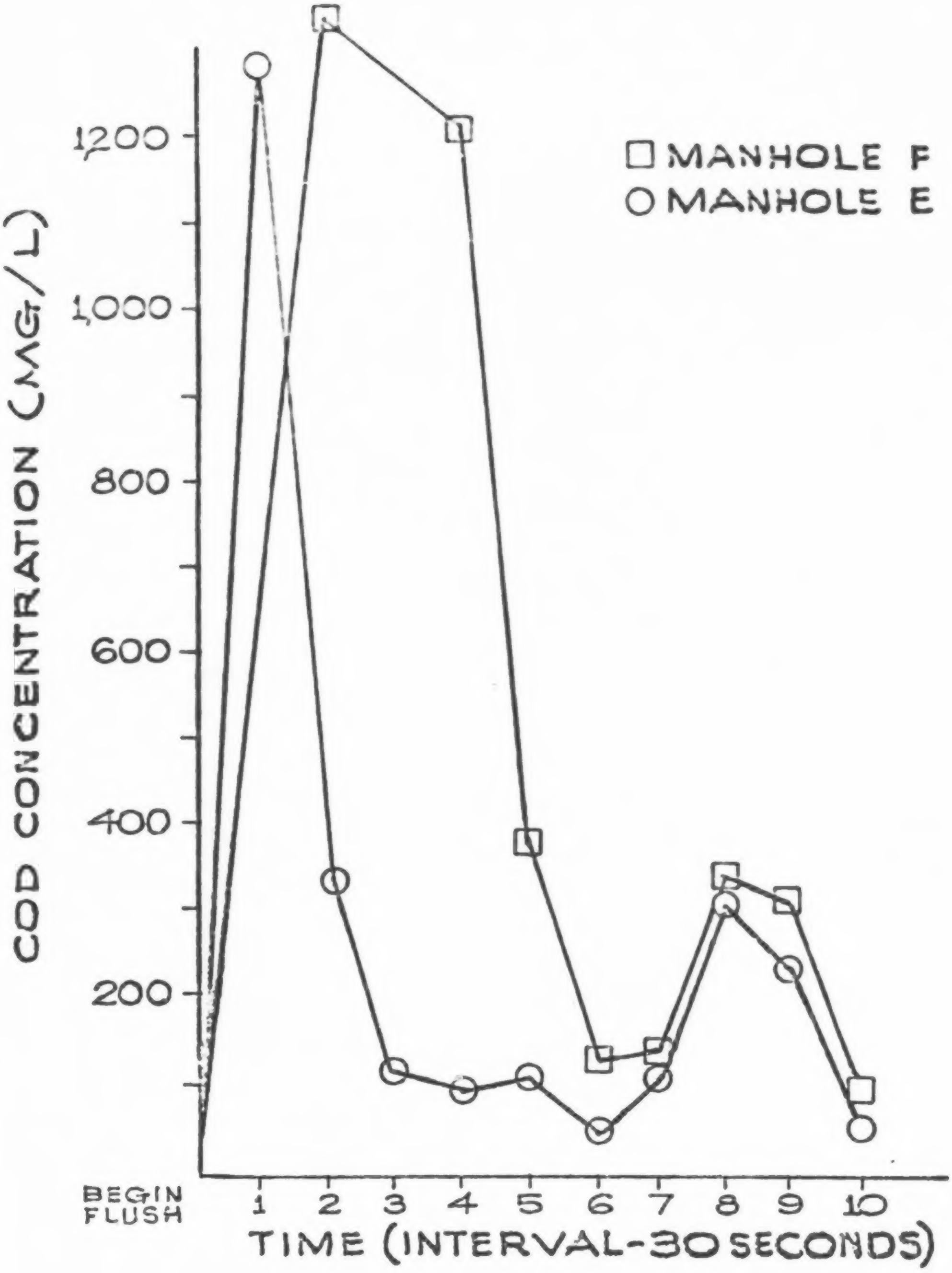


Figure 44.

Phase II - COD West 103rd Street

A standard fire hose was positioned in the street center at the crown of the road and the water was directed from the center to the gutter above the catch basin. The hydrant discharge was set as previously calibrated to allow a flow velocity of 3.7 fps. Samples (one per minute for ten minutes) were taken at both the catch basin mouth and at manhole F, after the initial crest passed.

Phase III Results

Figure 45 shows the large amount of pollutants occurring on the impervious surface. Unlike Phase I and II, there was no initial surge of this highly contaminated water. The concentration of pollutants in the samples taken downstream graduated to maximum after five minutes and then gradually leveled off.

Phase III, like Phases I and II, demonstrates a possible method for reducing pollution.

DISCUSSION

The study illustrated that, for combined sewers, a regularly conducted program of sewer flushing could greatly lessen the amount of pollutants reaching receiving waters via storm overflows.

The procedures used in Phase I appeared to be effective for quickly removing potential pollutants which accumulate in the sewers. This method, however, does not take into consideration problems resulting from urban runoff and from street pollutants.

Although it was more difficult to set up and expedite than Phases I or III, Phase II, using an inflatable plug, also appears to be an effective technique in quickly eliminating settled material in the sewer. Phase II also ignores street pollutants. Problems in Phase II, specifically with the sewer stopper, could reduce the plan's effectiveness as an operational tool. The "balloon-stopper" was difficult to remove from the sewer on one occasion causing a partial flow rather than the desired surge effect. It also collapsed prematurely on another occasion when water pressure caused the separation of the bladder from the vent tube.

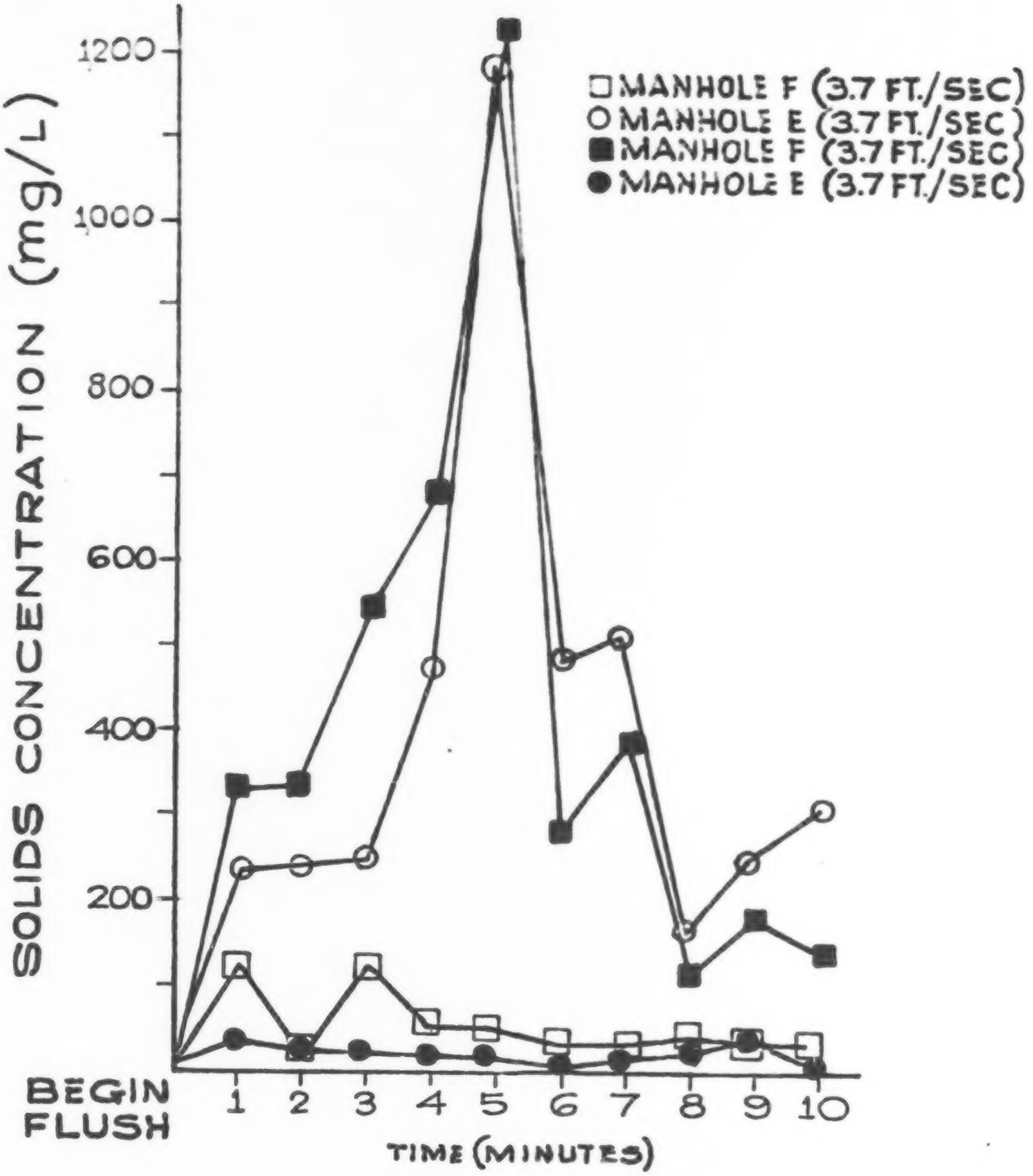


Figure 45.

Phase III - Suspended Solids
West 103rd Street

Removal of suspended solids and COD was not substantially better with an instantaneous release of water than with a continuous flow. Perhaps a greater hydraulic head would be more effective, but the depth of most residential area manholes would not be sufficient for a significant increase in depth than the depth used in this study. Phase II is the most time consuming because of operation problems. Phase II would be the least effective of the three methods tested.

The method used in Phase III may ultimately prove to be the best of the three techniques tested. It realistically combines the removal of street pollutants with sewer flushing and can easily be accomplished with a fire hose and hydrant or a tank truck. The manhours involved in actual sewer flushing using the methods delineated in Phase III are significantly less than the manhours needed for the other two operations. Consequently, Phase III is also the least expensive sewer flushing operation.

Another consideration is that the program will accomplish little if the streets and laterals are cleaned of pollutants only to have them resettled in larger trunk sewers. Such material could again be overflowed prior to treatment during high flow periods. To prevent such occurrences, several streets should be flushed simultaneously creating a flow of sufficient intensity to carry pollutants to the treatment plant.

Further studies should be made to determine how long sewers remain "clean" after flushing. If the solids build-up takes place in a matter of several hours after flushing, flushing frequency would have to increase beyond economic feasibility in order to achieve satisfactory results in reducing pollution levels.

In Cleveland or other municipalities with a large number of overflows which release pollutants frequently, sewer flushing methods could be useful in abating pollution and could most likely be included, dependent upon some further study, into the City's routine street cleaning.

SECTION VIII

SEWER OVERFLOW REDUCTION USING POLYMERS

INTRODUCTION

The greatest source of lakefront pollution in Cleveland is the storm overflow of combined sewers. These storm overflows can pour millions of gallons of raw untreated sewage into Lake Erie in a short period of time. Raw sewage in the Westerly Interceptor, the sewer studied in this test, begins to overflow weirs after a rainfall with a rate of 0.02 inches per hour. Hydrographs developed for a pollution abatement survey (Appendix G) indicate that rainfall in excess of 0.02 inches per hour occurs about 540 hours a year. Five-hundred forty hours of combined sewage overflow is well beyond the assimilative capacity of the lakefront waters.

In an effort to reduce the amount of combined sewer overflow, it was hypothesized that an addition of polyelectrolyte polymers into a combined sewer above an overflow would increase the velocity of the sewage. A sewer, then, would be able to handle a larger volume of sewage before overflowing. With the major source of beach pollution reduced, the beach clean-up would proceed much better.

In previous studies, the introduction of polyelectrolyte polymers into flowing sewers has been shown to significantly increase the water's velocity. The Western Company[4] under contract from the Federal Water Pollution Control Agency (presently EPA) did extensive testing to determine the effect of polyelectrolyte addition into a sewer. Experiments were conducted to determine what type of polyelectrolyte and at what concentration would yield maximum benefits. In their testing, the Western Company experienced flow increases up to 2.4 times the normal flow. To date, however, work with polymers to increase sewage flow has been done with closed conduits flowing full. It is the purpose of this section of the Demonstration of Interim Techniques for Reclamation of Polluted Beachwater to determine if the polymer addition concept will also work in a non-surcharged sewer. This polymer program had been conceived and developed by Havens and Emerson, Ltd., Consultant Engineers.

CHOICE OF TESTING SITE

A segment of the Westerly Interceptor, which parallels the lakeshore of Cleveland's West Side and is a major contributor of storm overflows, was chosen for the site for this test. This site, downstream from the Edgewater Beach overflow, was previously used by Havens and Emerson and the Bureau of Wastewater Quality Control for sewer flow studies. This segment of the Westerly Interceptor is brick-lined and has a 6 foot, 3 inch diameter. The normal dry weather flow is 15.07 CFS (9.7 MGD) with a daily peak at 18.1 CFS (11.7 MGD). Overflow at the overflow chambers occurs when the sewage flow rate is at 20 CFS (12.9 MGD) (Appendix G).

POLYMERS SELECTED

Based on monies allocated for this project, 50 mg/l additions of polymer would be the maximum dosage that would be economically feasible. According to the Western Company's results, an addition of 50 mg/l polymer could increase the sewage flow by about 50%. With the conclusions from Western's study in hand, five polymer manufacturers were approached regarding the use of their polymer in this study. Each polymer manufacturer was asked to recommend a polymer for the test and, if it differed from the ones selected by the Western Company, explain the change. As a result, several of the manufacturers tested their polymer in test loops and provided Havens and Emerson and the Bureau of Wastewater Quality the most appropriate polymer. Table 4 lists the polymers selected.

METHOD OF POLYMER ADDITION

Each polymer was mixed with potable water. A mixing device was used consisting of a chemical feeder, a vibrating tray, a mixing tank, and a water supply. The feeder was a standard hopper with a variable auger feeder in the "V" of the hopper. The auger delivered the dry polymer to the vibrating tray which had a variable frequency of oscillation. The purpose of the vibrating tray was to keep the dry powder (polymer) separated and allow it to drop off the tray as individual particles. The water supply was distributed over a flume and discharged into the mixing tank under the vibrating tray so that the falling particles of polymer would be intercepted by the water and wetted thoroughly before reaching the mixing tank. The water flow was controlled by meters which were provided to allow the preparation of accurate mix concentrations. The mixing tank, powered by a Lightning Mixer, was approximately 3 feet in diameter and 3 feet deep.

The diluted polymer solution was transferred by a positive displacement pump to one of two 6,000 gallon holding tanks.

Table 4. POLYMERS SELECTED FOR INVESTIGATION

Manufacturer	Used in This Study	Used by Western Co.	Chemical	Remarks
American Cyanamid	[illegible]	——	Anionic, high molecular weight	Similar to Dow A11 and Calgon ST269 except it has molecular weight
Calgon	ST269	B212	Copolymer (Acrylic/ acrylamide)	Equivalent
Dow	AP-30	A11	Copolymer (Acrylic/ acrylamide)	Equivalent
Stein-Hall	CC-1	——	Anionic, high molecular weight	Recommended by the Halliburton Co. based on test loops
Union Carbide	Polyox Coagulant #58-302	Polyox Coagulant #58-302	Polyethylene Oxide	

The solution was generally used within 8 hours after preparation. Additional mixing was provided in the holding tanks by air pumped through an "X" shaped diffuser pipe in the bottom of each tank. When a sufficient quantity of mix was prepared, the water meters were read and the exact polymer concentration was computed. The holding tanks were calibrated and staff gauges were installed to further check the volume. Polymer manufacturers' representatives inspected and approved the solutions and the method of mixing and dissolving the polymer prior to use.

METHOD OF TESTING

At the time the polymer was to be tested, measurements were made to determine sewage depth and if the sewage was flowing at a constant rate. The velocity of the sewage was determined by measuring the depth of flow and reading the velocity from previously developed rating curves (Appendix G). The depth measurements were also taken at the monitoring station. The depth was monitored by a nitrogen-type bubbler depth gauge and by means of a cloth measuring tape. If the polymer additions were to be beneficial, the results should be a decrease in water depth at the monitoring point.

The decrease in depth would be caused by an increase in velocity since the flow of sewage into the testing site was constant. In conjunction with the addition of polymers, a tracer dye, Rhodamine B, was added to the sewer at a constant rate. The concentration of tracer dye in the sewage was constantly analyzed during the addition of polymers by use of a fluorometer. Changes in the dye concentration would indicate a change in the flow of the sewage entering the test area. The test was performed during periods of dry weather flow so no changes were noticed. Typical velocity for sewage flow during the testing was 3 ft/sec. The polymer addition time and length of observation was 40-60 minutes.

RESULTS

The site was prepared early in the summer of 1969. During July, two dry runs were made to test the equipment and familiarize personnel with its operation. Actual polymer testing began on July 31. Eleven separate tests were conducted. The results are summarized in Table 5.

No increase in flow depth and velocity could be measured with any of the polymers tested. With the exception of Union Carbide Polyox Coagulant, each polymer was tested at

Table 5. THE EFFECT OF VARIOUS POLYMERS ON THE FLOW VELOCITY IN A PARTIALLY FILLED 6' 3" INTERCEPTOR SEWER.

Polymer Mfgr. & Run #	Sewage Discharge (mgd)	Polymer Discharge (gpm)	Dilution Water (gpm)	Polymer Conc. in Sewage (mg/l)	R_e (10^{-5})	Velocity of Flow (fps) Before Polymer	Velocity of Flow (fps) After Polymer
1. Calgon ST269	14.5	58	none	16	3.6	2.9	2.9
2. Calgon ST269	14.5	75	none	28	3.6	2.9	2.9
3. Dow A22	14.4	58	none	58	2.8	2.8	2.8
4. Dow A22	13.5	86	none	72	2.8	2.8	2.8
5. American Cyanamid [illegible]	14.8	88	none	9	2.8	2.8	2.8
6. American Cyanamid [illegible]	14.8	89	none	44	2.8	2.8	2.8
7. Am.Cyanamid	14.8	286	78	126	2.8	2.8	2.8
8. Stein-Hall CC-1	20.8	229	62	65	3.3	3.6	3.6
9. Stein-Hall CC-1	20.8	258	138	82	3.3	3.8	3.6
10. Stein-Hall CC-1	20.6	264	138	190	3.3	3.6	3.6
11. Union Carbide	20.8	208	none	208	3.3	3.6	3.6

Reynolds Number - Form used for the Calculations: $R^* = \frac{DV}{\nu}$ Where D = diameter (ft.)

V = velocity (ft/sec.)

ν = kinematic viscosity, for water @ 70°F 0.0000105 (ft²/sec.)

For open-channel flow, D = 4 X hydraulic radius least two times with a different polymer concentration being used for each test. It is significant to note that a polymer concentration as great as 100 mg/l (run #10, Stein-Hall CC-1) was ineffective in measurably affecting the flow velocity. For runs #7-10, the water supply at the polymer feeder station was used for additional dilution and dispersion of the polymer. The additional dilution water did not produce improvement in the polymer effects on the sewage flow.

DISCUSSION

There are two different explanations for the failure of the polymer test. The first explanation is that the principles underlying velocity increases in previous studies do not fit this study's situation. The experiment done by the Western Company was conducted in a 6 inch PVC pipe and in a 24 inch concrete pipe. In our study, testing was done in a 6 foot, 3 inch brick-lined sewer. Since the exact mechanism for hydrodynamic drag reduction is not known, it is possible that the passage of water over a rough brick surface could cause sufficient turbulence to outweigh any reduction in "apparent pipe friction" caused by the polymer. In other studies, surcharged pipes with water flowing full and at high velocities were used. In the present study, the depth of flow was 1.8 to 2 feet in a 6 foot, 3 inch sewer, and the velocity of sewage was generally much lower than in previous studies. However, in a more recent study, the University of Massachusetts did the test in a 24 inch cast iron pipe, flowing partially full with velocity at 2 to 3.5 feet/second. The sewage velocity was increased by 100%.

A second explanation for the failure to see effects of polymer addition could be the fact that adequate time to consider all the nuances involved in the testing was not available.

1) The method of testing was chosen to "see" only large changes in sewer flow characteristics. An actual increase in sewer velocity could have occurred but not been noticed. Looking at Table 6, one can see that in run #7, using American Cyanamid 836A polymer, 265 gpm of dilution water were used. Two-hundred sixty-five gpm increases the sewage flow by 2.72%. This increase in flow could not be detected by the method of testing used.

2) The distance between the point of polymer addition and the monitoring stations was 1400 feet. This distance is too short to permit measurements to be sensitive enough to detect small flow changes, and,

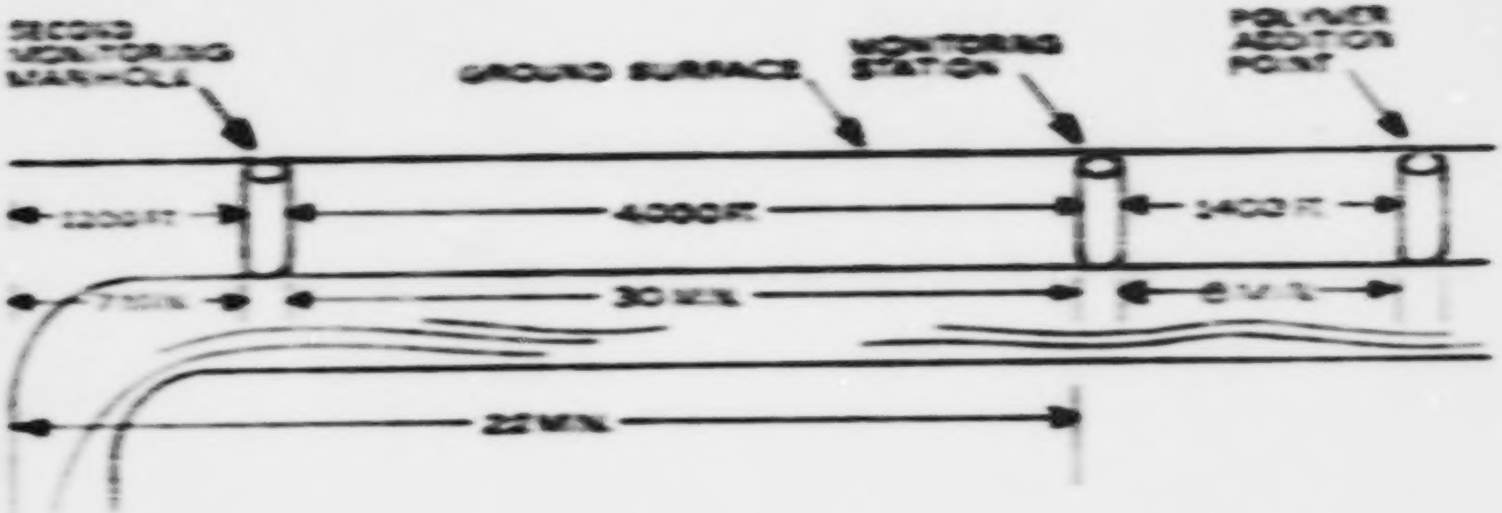

SECOND MONITORING MANHOLE
GROUND SURFACE
MONITORING STATION
POLYMER ADDITION POINT
1200 FT
4000 FT
1400 FT
7 MIN
30 MIN
6 MIN
22 MIN

Figure 46.

Profile of test segment of the Westerly Interceptor

3) It is conceivable that the duration of each polymer run wasn't long enough to "see" the effects caused by the polymer.

Using an average flow 3 fps, the polymer would take 8 minutes to reach the monitoring stations, 30 minutes to reach the second monitoring manhole (see Figure 44). Seven more additional minutes would be required for the polymers to reach a steep incline in the sewer where the sewage velocity is greatly increased. It is only until the polymer has reached this point that one should expect the sewage to begin a stage-like increase in velocity. To expect velocity increases before this time, one would have to envision the polymer ladden waters overtaking the water ahead, rushing over it in the fashion of a wave breaking on a beach. If, for discussion purposes, the flow increases to 4 fps, it would take another 22 minutes for this increase to be observed 5200 feet upstream at the monitoring station. The total time for the effects of the polymers to occur, then, would be an hour and seventeen minutes. The actual testing duration was only 40 minutes to an hour.

The concept of polymer addition to partially filled sewers to reduce storm overflow has merit. Polymer additions should be studied further to answer theoretical and practical problems associated with their use.

SECTION IX

DEBRIS REDUCTION USING SCREENING DEVICES

INTRODUCTION

Except in those beach areas which are continually "policed" during the bathing season, large quantities of trash can be found everywhere along the Cleveland shoreline. Much of these solids enter the lake from Cleveland's streams and sewer overflows where no treatment occurs. It has been the intent of this section of the project to provide in-stream removal of debris before it becomes a contributing factor to the degradation of Lake Erie and its shoreline.

In the original proposal for the Demonstration of Interim Techniques for Reclamation of Polluted Beachwater, Nine Mile Creek, Dugway Brook, and Edgewater Overflow were to receive some manner of screening to prevent trash and floating debris from being discharged into Lake Erie. However, due to inaccessibility of the outfall by land for servicing, and the lack of an alternate location, plans for installing screening devices at Dugway Brook and at the Edgewater Overflow were deleted from the project. Only Nine Mile Creek received a screening device for this demonstration project.

Nine Mile Creek drains five thousand acres of the northeast Metropolitan Cleveland area. The creek discharges a mean daily flow of seventeen cubic feet per second directly into Lake Erie. The mouth of the creek is approximately one mile west of White City Park, a major bathing area. Prevailing lake currents carry discharges from Nine Mile Creek to the beach at White City. A removable basket screening device was constructed (Figure 17) approximately three thousand feet from the mouth of this creek on the upstream side of an inverted syphon/culvert structure.

DEBRIS REMOVAL

To insure that the major portion of dry weather flow passes through the screen, a debris removal apparatus was positioned in the center of the stream bed, equidistant from the banks of the creek. Functioning as designed, the device would screen floating trash and debris from the creek as it flows through the screen. The site would be under surveillance to determine when the screen should be cleaned. Trash removal and cleaning of the screen was accomplished by manually hoisting the basket to the top of the culvert, where

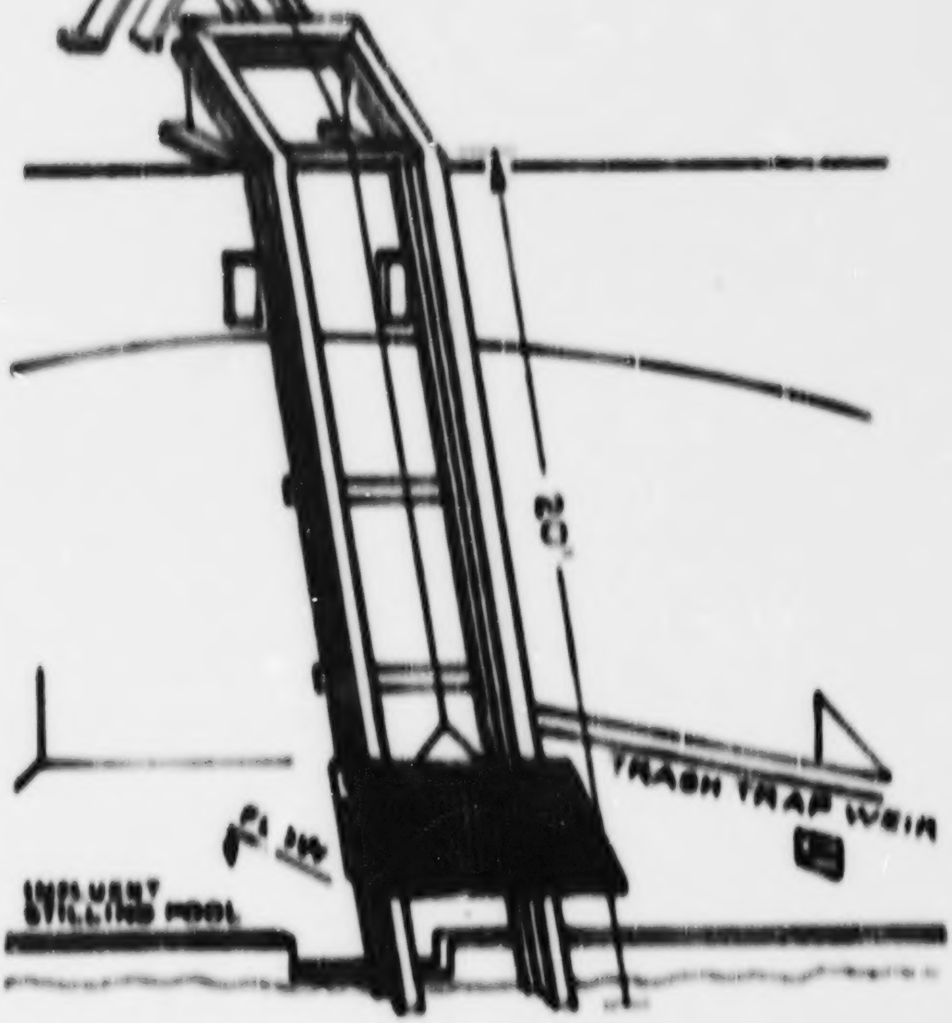

20'
TRASH TRAP WEIR
INFLUENT STILLING POOL

Figure 47.

Screening device Nine Mile Creek

debris could easily be unloaded and trucked away. After cleaning, the basket was lowered again to its original position.

OPERATION OF SCREENING DEVICE

Installation of the screening device was completed in the spring of 1969. It was not long afterward that heavy rains loaded the basket with an extraordinary amount of debris. During the first attempt at cleaning, it was discovered that a small hand operated winch was not capable of lifting the basket when filled to capacity. With assistance from the City of Cleveland's Sewer Maintenance Division, a mobile crane was used to remove the basket for cleaning. The net weight of the trash was found to be slightly under one ton and with the addition of the basket weight, the gross weight exceeded the winch's one ton capability.

This experience set precedence for future policy in the operation and maintenance of the screen device. The screen demanded more attention than was initially thought. The site had to be inspected at least daily to insure immediate cleaning before overloading of the basket could occur. The Bureau of Wastewater Quality continued in its surveillance role while sewer maintenance personnel, being better equipped for the job, assumed responsibility for cleaning the device. Subsequent cleanings presented fewer problems than the first attempt.

In the early spring months of 1970, when again large quantities of trash held along the banks by winter ice began washing downstream with the heavy rains, it was almost impossible to keep the screen cleaned. As quickly as the trash was removed, the basket would be full again. When the basket was allowed to sit full for any length of time, it acted as a dam to stream flow. The stream bed consisted of small lightweight stones which made it easy for the creek to change its course in the wide stream bed to one which carried most of the flow around the screen. Twice a mobile crane was used to dig a channel through the center of the stream bed in order to return the main flow to a course passing through the screen. After spring, when the large influx of trash had tapered off, there were no more problems of this nature.

EVALUATION OF REMOVAL PROGRAM

The screening device needed a cleaning about once every two weeks on a seasonal average and was in use from May to

September. The apparatus was found to be especially well adapted to catching leaves, rags, paper and small tree branches. Total trash removed from Nine Mile Creek by means of the screen over the two season period has been estimated at twenty tons. Cost for installation and maintenance of the screen has been approximately $4500 total for both years, or $225 per ton. The majority of this cost was in equipment. A breakdown considering only labor and maintenance costs yields a $74 per ton figure.

Because of the large amount of floating debris that is discharged into Lake Erie by streams, a pollution control program such as the screening operation is worthwhile. Automatic self-cleaning screening devices could have a marked impact on the reduction of lakefront debris.

SECTION X

DEBRIS AND SOLIDS COLLECTION

INTRODUCTION

Debris from natural origins and from negligent and intentional dumping of trash along the rivers and at numerous points along the entire shoreline of Lake Erie provides a continuing influx of water-borne debris into navigable waters and onto the beaches. The debris is a problem of safety and one of diminishing aesthetic beauty of our parks and waterways. Logs as large as four feet in diameter and fifty feet long have been seen floating down the Cuyahoga River. Pieces this large are a hazard to commercial as well as pleasure boating. Bathing beaches, covered with water-borne litter, are not conducive to safe recreation, nor does a pervading stench from decaying organic material promote popular or frequent utilization of public recreation areas.

Debris removal in a city as large as Cleveland is a major task. Approximately seven and a half miles of river, five miles of lakefront, and twenty acres of sandy beaches required intensive care. Clean-up operations were conducted from mid-June to mid-September for 1969 and 1970. Appropriate removal equipment was a prerequisite for an effective operation.

Beach maintenance is not something new to Cleveland as a result of this project. Debris removal operations have been an important part of summer recreation programs in the past. Clean-up was accomplished for the most part by hand and through the use of various mechanized equipment, none of which had been designed to function primarily as beach cleaning equipment. As the quantities of debris increased to what it has been in recent years, so too did the labor force and resulting costs required for clean-up. Results were less than desirable and the City began looking at alternate methods.

METHOD OF DEBRIS REMOVAL

A beach sanitizer (Appendix H) was purchased in 1969 for $25,000 to aid in the methodical and thorough debris removal at the two sandy beaches (White City and Edgewater) operated by Cleveland. The sanitizer aerates and sifts the sand while removing small debris of the type almost always found on the beaches. The sanitizer unit was operated in the morning before bathers arrived and was driven to the alternate beach in the afternoon. The beaches are approximately eleven miles apart. Two hours were required for transport of the sanitizer.

In operation, the sanitizer traversed the beach longitudinally on six foot centers, one pass each. A riffling attachment (scarifier) preceded the sifter unit. The loose sand and debris were forced onto a screen and separated by the action of rotating brushes. Material removed was transferred to dump trucks for disposal at a land-fill site.

To further enhance the maintenance of debris-free beaches City crews skimmed the floating debris from the water within the bathing enclosures. A small boat, equipped with nets and a five horse-power outboard engine, was used. The same crew also raked debris out of the water at the shoreline on occasion.

RIVER AND LAKEFRONT

During the 1969 season a scavenger boat was operated as it had been for the previous four years, primarily in the navigation channel of the Cuyahoga River. In 1970, a modified operation also covered the lake shore area between the two bathing beaches.

The purpose was to remove floating debris and, in the case of the river, oily wastes associated with the debris. The recovered material was carried to a land-fill for final disposal in 1969, and incinerated in 1970.

The boat used in 1969 was equipped with a fork-lift device for the debris and a belt-type surface oil separator. The separator (or skimmer) was not effective for oil slicks containing debris, but did operate well for a [illegible],000 gallon, Bunker C spill which had been contained by booms. The oil removal rate during this spill was 1000 gallons per eight hour day.

In 1970, the effort was extended to 61 days of operation using equipment of substantially different design. Debris removal at points of known entrapments along the lake shore was emphasized more than the previous year.

Concentration of debris were sufficiently large in most areas at the beginning of the season to require continuous daily operation. Two thirds of the way into the contract period the amount of debris was considerably diminished, and the contractor was directed to limit operations to only 2 or 3 days per week according to need. As a result, debris removal work was spread over a longer season and greater overall effectiveness was achieved from the operations.

Equipment used in the operation included: 1) a 55 foot steel-hulled fishing boat with a stern mounted 5 ton capacity crane featuring a clam shell construction of heavy industrial mesh and a load deck area aft of approximately 15 feet x 25 feet; 2) two large flat bed trucks, one which was equipped with a 5 ton capacity and hydraulic dump capability; and 3) a large dome-shaped drive-in incinerator.

A representative of the City was assigned to coordinate all phases of the operation with the contractor and supervise the work performed. The representative periodically accompanied the contractor during removal operations to view the progress in the operation as well as to implement ways to increase the efficiency.

In the initial stages of the operation, the contractor utilized two crews, one working from the shore with the truck mounted crane operating in locations inaccessible by water, and the other crew working from the scavenger boat. Accumulations of residual debris were removed from a number of lakefront marinas, a power plant water intake, the navigational channel of the Cuyahoga River and the Old Cuyahoga River Bed West. The debris removed consisted of drift from natural origin in the forms of trees, logs, limbs, and tree root masses, portions of deteriorated waterfront structures such as timbers and drift generated by human activity, such as tires (some with rims), oil drums, cable reels, mattresses, pallets and various kinds of small refuse. In addition, large quantities of Bunker C type oil which was released when a derelict ship sunk in the river were removed concurrently with accumulations of small debris, particularly in areas near the head of the navigational channel.

Although some trapped oil was removed with the debris, significant amounts of floating oil persisted and, in the future, other methods will be required for ultimate removal.

In 1979, the three-man boat was operated as follows: 1) the pilot maneuvered the boat into position; 2) the crane operator lifted the undisturbed debris from the surface by manipulating the controls of the fine screened pincer-like basket; 3) the load was deposited on the 15' x 25' deck area, and 4) a third crew man assisted by positioning the debris for pickup with the aid of a specially equipped pole. Excessively long pieces of driftwood, such as trees, were cut into approximately 15 foot lengths for convenience of handling at the offloading area by the dockside truck-mounted crane. The material was then hauled to the incinerator for final disposal.

This procedure varied considerably from previous years in two ways. First, only the larger pieces could be removed with the flat "basket" at the base of the lift-truck device, and second, the material was positioned and offloaded manually, with an additional step required to load it onto a truck for removal.

RESULTS

Beaches

Visual observations indicated that the beaches had been greatly improved from an esthetic viewpoint, and this was reinforced by comments from area residents. Appendix H contains the pertinent details for maintaining clean sand beaches. The cost of $76 per ton was considered reasonable for this type of operation.

The manual skimming efforts were not required frequently and the miniscule amount of material (on a tonnage basis) removed did not allow a meaningful cost benefit analysis. The crew members involved in these efforts were sharing time from other auxiliary operations and kept no explicit time record. The wages and operating expenses for this activity were estimated to be approximately $100 per month.

River and Lakefront

The Cuyahoga River was visibly improved in appearance and from a safety standpoint. Fewer telephone complaints were received in 1970 than in any of the last ten years.

A comparison of aerial photographs taken over the past years showed that the river surface had less visible debris in 1970 than in any of the past fifteen years. Oil accumulations of the type which caused the river to burn in 1969 were never able to form during the 1970 operating period.

In 1969, the scavenger boat operated for forty days and removed 216 net tons at an average cost of $80.55 per ton.

Table 6. DEBRIS AND OIL REMOVAL OPERATION

Inclusive Working Period	Total Days	Total Est. Wet Tonnage Removed	Average Est. Tons Removed Per Day	Avg. Cost Per Ton Removed
8/19/65 - 12/20/65	40	460	11.5	$ 33.48
6/13/66 - 11/19/66	50	420	8.4	67.56
6/14/67 - 11/24/67	50	419	8.4	66.75
6/14/68 - 11/14/68	60	489	8.2	69.71
7/01/69 - 11/02/69	60*	375	6.3	69.60
7/10/70 - 10/16/70	61**	775	12.7	53.00

* 20 days of contract were expended on oil spill clean up without concurrent debris removal and unrelated emergency work.

** Actual days on the water.

The type of equipment used on the 1970 season increased the efficiency by more than 100%, based on the 1969 costs. This was due, in part, to the fact that floating material was removed both from the land and water side of the river. Debris was removed from many areas which had previously gone untouched due to their locations and the tonnage removed was doubled compared to the previous year.

Table 6 compares the debris removal for the six years since this type of activity was begun.

SECTION XI

MISCELLANEOUS IMPROVEMENTS

INTRODUCTION

During the Beach Project field investigators of the Bureau of Wastewater Quality did extensive searches through sewers to look for irregularities in sewer design or sewers in need of repair. These problems in sewers could greatly contribute to the amount of pollution pouring into Lake Erie. The searches resulted in the discovery of three major problems that directly affected the quality of the lake. Improvements were then proposed to remedy the problems. These improvement proposals were:

1) Shaw Brook Diversion and Interception
2) Nine Mile Creek/Hilltop Sanitary Sewer regulator improvement
3) Dugway Brook Pump Station

DISCUSSION

Shaw Brook flows from the southerly boundary of East Cleveland, through Cleveland Heights and Cleveland, and enters Lake Erie at the Bratenahl shoreline. The stream is polluted by both sanitary and industrial wastes. For the past eight to ten years, a portion of the dry weather flow has been diverted into the Easterly interceptor sewer and transported to the Easterly Sewage Plant. The structure frequently becomes clogged, causing it to malfunction. To alleviate this problem a new regulator has been designed which would divert up to fifty times the dry weather flow into the interceptor. Stream flow diversion will be accomplished by a motor operated sluice gate regulated by a monitoring unit within the interceptor sewer. The Shaw Brook rehabilitation program will reduce the pollution load of Shaw Brook to Lake Erie by directing all of the dry weather flow and a portion of the storm flow from the stream's drainage area into the Easterly interceptor sewer. The plan will also alleviate existing drainage and maintenance problems in the area. The project is ready to bid and it is anticipated that easements will be obtained by March 15, 1971 with construction to begin June, 1971.

The Nine Mile Creek regulator diverted combined sewage from the Hilltop sanitary sewer to Nine Mile Creek whenever the rate of flow was more than 15 MGD; daily peak flows nearly always exceed 15 MGD in any weather. As a result, the regulator was spilling sewage into the creek almost daily. A diversion weir has been installed to increase the capacity of the combined sewer from 11 MGD to 27 MGD. Storm flows which exceed 27 MGD are still bypassed to the creek. The problem of bypassing from the Hilltop area to Nine Mile Creek will not be remedied until adequate sewers are built to carry the sanitary sewage and the excessive storm water infiltration to the treatment plants.

A pumping station on Dugway Brook was designed to divert dry weather flow from the brook to the Cleveland Easterly interceptor and subsequent treatment. The new Dugway Brook pumping station was designed to intercept flows ranging from a minimum of 1.5 MGD to 25 MGD, equal to seven times the average dry weather flow. The pumping station was needed because the brook received sanitary wastes from suburban residential areas and by overflow from the combined sewer systems of Cleveland and East Cleveland, also over eighty overflows discharge sanitary sewage into Dugway culverts either because of plugged sanitary lines or as a consequence of storm flows. Installation of the pumping station will increase the total quantity of wastewater which receives sewage treatment and protect the lower reaches of the stream from pollutants.

SECTION XII

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Table A-1. WHITE CITY BEACH 1969
Summary of Bacteriological Data - 6/28 thru 7/29
Prior to Chlorination and Enclosure

Bacteria	Number of Days Sampled	Arithmetic Mean/100 ml	Geometric Mean/100 ml	No. of Days Bacteria > 10000/100 ml
Total Coliform	25	17,200	6,800	8
Fecal Coliform	25	2,400	1,200	---
Fecal Streptococci	24	150	50	---

Table A-2. WHITE CITY BEACH 1969
Summary of Bacteriological Data - 5/28 thru 6/23
Prior to Chlorination and No Enclosure

Bacteria	Number of Days Sampled	Arithmetic Mean/100 ml	Geometric Mean/100 ml	No. of Days Bacteria > 10000/100 ml
Total Coliform	20	30,100	18,600	8
Fecal Coliform	20	2,200	950	---
Fecal Streptococci	20	370	120	---

Table 6-3. WHITE CITY BEACH 1969
Summary of Bacteriological Data - July 27 thru August 30
Manually Chlorinated and Enclosed

Total Sampling Period				Sampling Times When Residual Chlorine Inside Barrier > 0.1ppm		
Location & Bacteria	No. Times Sampled	Arithmetic Mean/100 ml	Geometric Mean/100 ml	No. Times Residual Chlorine 0.1ppm	Arithmetic Mean/100 ml	Geometric Mean/100 ml
Inside						
Tot. Coliform	[illegible]	5,300	2,200		650	170
Fec. Coliform	[illegible]	700	120	10	150	28
Fec. Strep.	[illegible]	29	22		2	1
Outside						
Tot. Coliform	[illegible]	16,083	3,300		----	----
Fec. Coliform	[illegible]	3,300	520	---	----	----
Fec. Strep.	14	120	26		----	----

Table 6-4. WHITE CITY BEACH 1969
Summary of Bacteriological Data - June 22 thru August 29
Automatic Chlorination Facility Operating and No Enclosure

Total Sampling Period				Sampling Times When Residual Chlorine Inside Barrier > 0.1ppm			
Location & Bacteria	No. Times Sampled	Arithmetic Mean/100 ml	Geometric Mean/100 ml	No. Times Residual Chlorine 0.1ppm	Arithmetic Mean/100 ml	Geometric Mean/100 ml	Times Bact. 1000/100 ml
Inside							
Tot. Coliform	87	22,300	1,500	36	800	182	29
Fec. Coliform	87	7,100	120		190	30	
Fec. Strep.	[illegible]	170	30		60	24	
Outside							
Tot. Coliform	[illegible]	25,400	10,600		13,700	7,000	1
Fec. Coliform	[illegible]	7,300	2,000		4,600	2,300	
Fec. Strep.	60	200	50		120	50	

Table A-5. EDGEWATER BEACH 1969
Summary of Bacteriological Data - July 3 thru August
Beach Enclosed - Manual Chlorination

Total Sampling Period				Sampling Times When Residual Chlorine Inside Barrier ≥ 0.1ppm		
Location & Bacteria	No. Times Sampled	Arithmetic Mean/100 ml	Geometric Mean/100 ml	No. Times Residual Chlorine ≥ 0.1ppm	Arithmetic Mean/100 ml	Geometric Mean/100 ml
7/9 - 8/5						
Inside Curtain						
Tot. Coliform	33	75,000	19,000	5	3,300	1,000
Fec. Coliform	33	7,100	1,400		71	31
Outside Curtain						
Tot. Coliform	33	58,000	11,000	1	---	---
Fec. Coliform	33	2,400	600		111	111

Table A-6. EDGEWATER BEACH 1969
Summary of Bacteriological Data 8/8 - 8/29
Beach Enclosed and Automatic Chlorination Facility Operating

Total Sampling Period				Sampling Times When Residual Chlorine Inside Barrier ≥ 0.1ppm			
Location & Bacteria	No. Times Sampled	Arithmetic Mean/100 ml	Geometric Mean/100ml	No. Times Residual Chlorine ≥ 0.1ppm	Arithmetic Mean/100ml	Geometric Mean/100ml	No. Times Bacteria ≥1000/100ml
Inside Curtain							
Tot. Coliform	26	6,800	730	20	1,800	540	22
Fec. Coliform	26	1,300	140		500	120	
Outside Curtain							
Tot. Coliform	26	20,100	1,200	20	9,000	1,200	22
Fec. Coliform	26	1,800	120	20	1,800	100	

Table 4-7. EDGEWATER BEACH 1969
Summary of Bacteriological Data - 6/23 - 9/4/69
Not Enclosed and Not Chlorinated

Bacteria	No. Days Sampled	Arithmetic Mean/100 ml	Geometric Mean/100 ml	No. Days Bacteria > 1000/100 ml
Total Coliforms	48	11,400	7,300	9
Fecal Coliforms	48	1,700	700	---
Fecal Streptococci	48	[illegible]	300	---

Table 4-8. EDGEWATER BEACH 1969
Summary of Bacteriological Data - 5/23 - 7/2/69
Prior to Chlorination and Enclosure

Bacteria	No. Days Sampled	Arithmetic Mean/100 ml	Geometric Mean/100 ml	No. Days Bacteria > 1000/100 ml
Total Coliforms	31	36,700	14,200	2
Fecal Coliforms	31	5,100	1,100	---
Fecal Streptococci	31	1,300	200	---

Table A-9. WHITE CITY BEACH FECAL COLIFORM DENSITY

Month	Number of Samples Taken	Geometric Mean of all Samples	No. of Samples Over 400	% Samples Over 400 per 100ml	Location Relative To Barrier
5/29 thru 6/21, 1970	126	900	141	81%	
6/22 thru	154	54	17	11.0	Inside
6/30, 1970	44	410	19	61.0	Outside
7/1 thru	269	23	42	14.8	Inside
7/31, 1970	141	260	85	60.0	Outside
8/1 thru	190	8	11	8.7	Inside
8/31, 1970	120	340	53	48.0	Outside
9/1 thru	91	7	7	9	Inside
9/7, 1970	13	325	9	42	Outside

Table A-10. EDGEWATER BEACH FECAL COLIFORM DENSITY

Month	Number of Samples Taken	Geometric Mean of all Samples	No. of Samples Over 400	% Samples Over 400 per 100ml	Location Relative To Barrier
5/29 thru 6/24, 1970	266	2520	111	67	
6/25 thru	150	78	40	22	Inside
7/24, 1970	126	56	35	30	Outside
7/25 thru	110	26	37	24.0	Inside
8/31, 1970	113	108	26	15	Outside
9/1 thru	73	36	20	15	Inside
9/7, 1970	41	126	11	26	Outside

POLLUTION BARRIERS

FIGURE A-1

INSTALLATION OF BARRIER AT WHITE CITY

FIGURE A-2

EDGEWATER BEACH WITH SUBMERSIBLE FLOTATION

FIGURE A-3

WHITE CITY BEACH SHOWING THE PONTOON FLOTATION

APPENDIX B

FISH KILL AT EDGEWATER BEACH

As a result of the initial "shock" chlorination of the Edgewater Beach swimming area on July 3, 1969, large numbers of fishes were killed. Seventy-four hundred gallons of sodium hypochlorite solution (15 trade percent available chlorine) plus 500 pounds of dry calcium hypochlorite (70% Ca (OCl)$_2$ by weight) were discharged directly into the swimming area producing a residual chlorine level of 0.5 ppm.

As seen in Table 1, nine species of fishes were observed killed by the hypochlorite. By far the greatest number of fishes killed were spottail shiners. In addition to the fish species killed, large numbers of sludge worms and the majority of the attached algae growing on the east stonewall were also killed, adding to the fish clean-up problems.

A laboratory experiment was conducted to determine the effect of varying levels of residual chlorine on fish. A common lake species, the emerald shiner (*Notropis atherinoides*) was chosen as the test species. Six fish were introduced into each concentration of residual chlorine. Several test runs were conducted. The average lethal dose for 50% fish kill (TLm) is graphed in Figure B-1. As indicated in the laboratory and also at Edgewater Beach, a level of 0.5 ppm residual chlorine was fatal to fish within twenty minutes. As shown in Figure 1, a concentration of 0.1 ppm residual chlorine was fatal to the fish after approximately 5 hours contact time.

Table B-1. FISH KILL AT EDGEWATER BEACH
JULY 3, 1969

COMMON NAME	SCIENTIFIC NAME	APPROXIMATE NUMBERS
Small Mouth Bass	*Micropterus dolomieui*	10
Yellow Perch	*Perca flavescens*	100
Freshwater Drum	*Aplodinotus grunniens*	500
Carp	*Cyprinus carpio*	300
Goldfish	*Carassius auratus*	300
Bluntnose Minnow	*Pimephales notatus*	500
Bluegill Sunfish	*Lepomis macrochirus*	300-400
Spottail Shiner	*Notropis hudsonius*	10,000

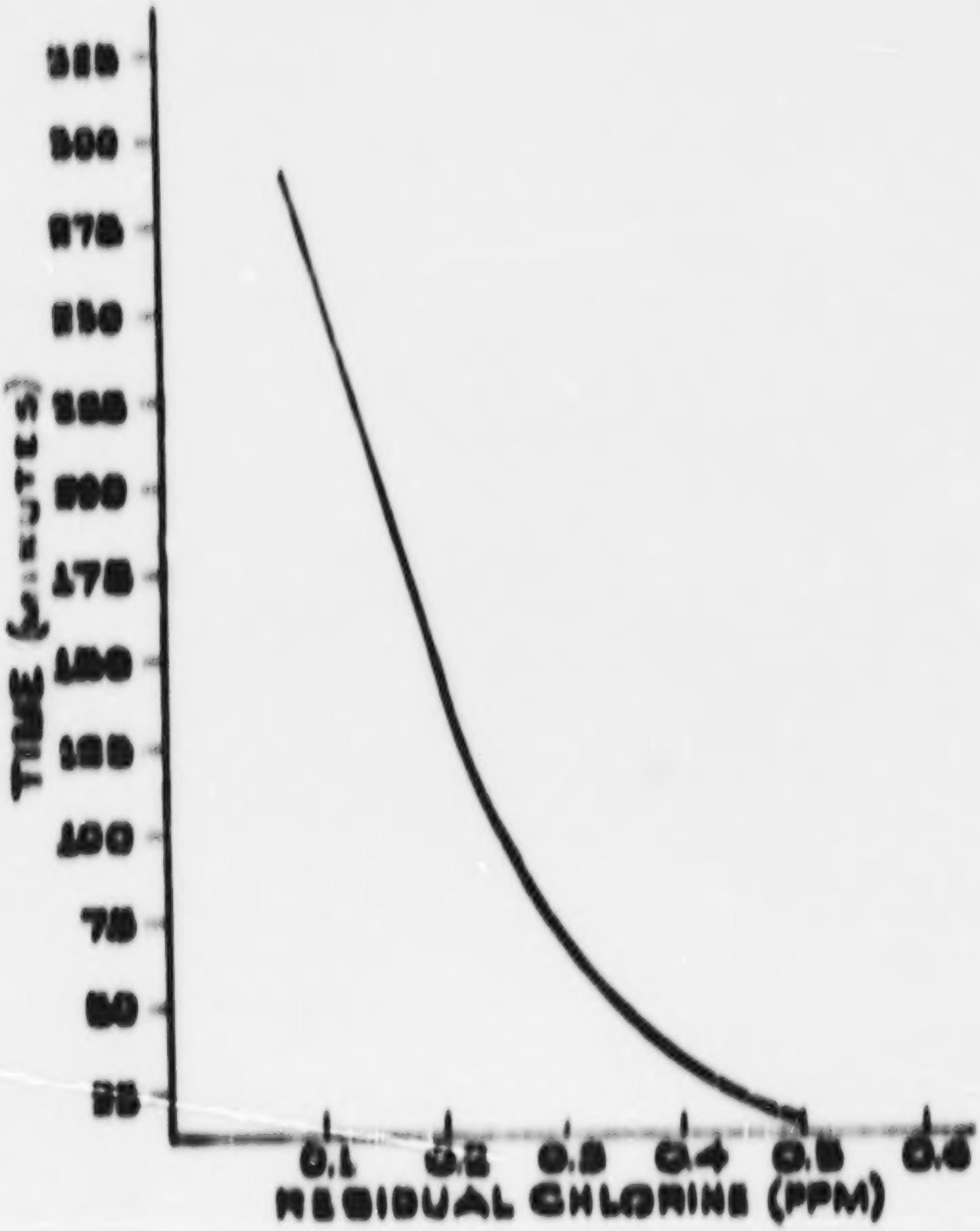

Figure B-1.

TLM of residual chlorine
on the Emerald Shiner

APPENDIX C

DESCRIPTION OF HYPOCHLORINATION FACILITIES

WEST 117TH STREET OVERFLOW HYPOCHLORINATION FACILITY

This station had a resistance level control with four levels. When the flow in the sewer reached the lowest sensor, it activated the first motor valve. As the level in the sewer increased, each motor was activated in turn. Thus the higher the flow through the sewer, the more hypochlorite was fed into it by gravity flow. There was a running time meter on the first motor valve to record "on" time. This station was equipped with four Ramcon Corporation [illegible] valve motor type pumps which provided good service. Only one minor problem was encountered in the operation of this station. The fourth motor valve was found defective and was replaced. Other than frequent emptying of the tank under high flow conditions, this facility operated continuously throughout the summer.

Figures C-1 and C-2 show a typical installation of a chlorination facility. Figure C-1 is an exterior view of a chlorination station housing tanks, feed pumps, controls and miscellaneous equipment used by the field crews. Figure C-2 shows a reinforced plastic holding tank for 15% hypochlorite solution was installed in each station. The proportioning hypochlorite feeding device in the foreground is one of several used in the program. Some were manually controlled while others were controlled by the stream flow rate via depth sensing devices.

EDGEWATER OVERFLOW HYPOCHLORINATION FACILITY

This station had a bubbler type gauge which was located at the outlet of the overflow sewer. The level gauge activated a variable speed motor controller which, in turn, drove a motor which turned a Roto Dip feeder. It was found that the Roto Dip feeder operated only 5 to 6 cycles during a period of overflow. The variable speed motor controller was found to be at fault and was replaced. Although no further breakdowns occurred, the system remained inoperative. It was then discovered that the bubbler level gauge had been displaced to the point where it would not operate. For the entire summer, the 700 gallon tank had to be filled only once. Thus the station was not considered to be operative for the summer.

WESTERLY OVERFLOW HYPOCHLORINATION FACILITY

This station had a bubbler type level gauge which activated a potential varying device, which, in turn, operated a peristaltic pump. At this station, the peristaltic pump was the only problem. The action of the pump rollers on the flexible tubing wore out the tubing rapidly. At maximum pump speed, the tubing was found to last only four hours. Due to a number of breaks in the tubing from the rotation wear, the entire pump was badly damaged by corrosion. Because of the large maintenance cost and the danger of uncontrolled spills when the tubing breaks, a different type pump was suggested. A feed pump equal to the Vanton Flex-I-Liner was suggested. Vanton size 10 or equivalent would meet all the requirements.

This pump would be run off the same variable speed D.C. motor as the peristaltic with small alterations in the mounting plate. A counter would be mounted on the motor to count the rpm to determine the amount of liquid pumped. The cost of the Vanton size 10 is $227 complete. This pump should give much better service. This station had a tank of 700 gallon capacity, which was found to be insufficient to chlorinate the overflow of an extended storm. However, except for frequent changes of the pump tubing, this installation operated throughout the summer.

HYPOCHLORINATION FACILITY AT DUGWAY BROOK

This station had a Fischer Porter dual flow sensing and feed system. It had low and high flow valves which were operated by a ratio determining magnetic flow meter. Because of the arrangement of pipes, meters, valves, and the tank, the head loss in the dual system were great enough to allow only 75% of maximum expected flow to the hypochlorite solution. Several problems were encountered in the maintenance of this station. A dirty batch of hypochlorite solution was introduced into the tank, which twice completely blocked the magnetic flow detector. The flow system had to be thoroughly cleaned before operation could be restored. Once the compressor which supplied compressed air to the pneumatic controls had a period of shutdown due to low oil level. During periods of high flow, the 1000 gallon tank was emptied rapidly, thus allowing portions of the overflow to reach the lake unchlorinated. However, despite the frequent shutdowns, this station was considered operative throughout the summer.

SHAW BROOK HYPOCHLORINATION FACILITY

Shaw Brook had a Fischer-Porter level sensing and flow control system. This system had only a low level control valve. This station was maintenance free for the summer, except for minor adjustments of the flow control valves. The 700 gallon capacity tank was found to be insufficient even though Shaw Brook has a comparatively low flow.

HYPOCHLORINATION FACILITY AT NINE MILE CREEK

Nine Mile Creek had a Fischer-Porter dual flow sensing and feed system. This station, as the one at Shaw Brook, had a bubbler type level gauge. Maintenance problems at this station were few. Once, the system had to be completely recalibrated when it was discovered that the low flow control regulator would not close to the ratio set by the flow meter. Here, as at Dugway, the arrangements of meter, tank, piping and valves would allow only 75% of expected maximum flow. Even though there was a greater flow in Nine Mile Creek, the hypochlorite tank, having a capacity of 1700 gallons, did not empty as rapidly as did those on the other two streams.

EASTERLY OVERFLOW HYPOCHLORINATION FACILITY

This station had a sonar gauge level sensing device built by C.B. Stevens and Company. The level gauge activated an Autocon pump control system which controls a peristaltic pump identical to the pump at the Westerly Hypochlorination Facility. Due to problems in the sonar level sensing device, this facility was never in proper operation.

CHLORINATION OF THE SWIMMING AREA OF WHITE CITY BEACH

Two attempts were made to operate the small Varistaltic Pump for continuous feeding of chlorine into the White City swimming area. Each attempt met with failure due to Tygon tubing fatigue and an inevitable uncontrolled spill of sodium hypochlorite within 1 to 20 hours of operation. An attempt was then initiated to feed the chlorine by gravity flow. A spill occurred within 48 hours due to the fact that a portion of the line had copper fittings rather than the specified PVC. This was readily corrected and swimming area chlorination was resumed by use of gravity flow for the balance of the season.

feeding by gravity flow proved to be relatively trouble free until a load of unfiltered sodium hypochlorite was delivered. Since that date, the system required almost daily attention owing to frequent flow reduction or stoppages from clogging action. In an effort to correct this situation, a [illegible] inch PVC line was later installed to the [illegible] inch PVC. This only partially alleviated the problem since the line continued to need frequent attention to combat the residual foreign matter restrictions and stoppages.

CHLORINATION OF THE SWIMMING AREA OF EDGEWATER BEACH

Edgewater Beach station's sole function was to chlorinate the Edgewater Beach area.

The system had two BIF Chem-O-Feeder Diaphram pumps which worked very satisfactorily for the entire season.

The sodium hypochlorite has done a considerable amount of corrosion damage to the chemical pumps and to the manual regulator valves. The entire system would have had to be reconditioned in order to be used for another season.

Table C-1. HYPOCHLORINATION STATIONS AND TANK CAPACITIES

1.	Easterly	3700 Gallons
2.	Nine Mile Creek	2700 "
3.	Shaw Brook	700 "
4.	Dugway Brook	3700 "
5.	Westerly	700 "
6.	Edgewater Overflow	700 "
7.	West 117th Street	700 "
	Total Capacities:	12,900 Gallons

Table C-2. USAGE OF SODIUM HYPOCHLORITE

Station	Gallons	Cost At 15¢/gal	Gallons	Cost At 15¢/gal
W.117th Overflow	2800	$ 420.00	7030	$ 1084.01
Edgewater Overflow	900	135.00	6450	991.74
Westerly Overflow	5600	840.00	6820	952.46
Easterly Overflow	0			
Dugway Brook	1800	270.00	32070	4507.00
Shaw Brook	700	105.00	3480	475.74
Nine Mile Creek	3700	555.00	37660	5781.13
Edgewater Beach	63280	9492.00	95910	15111.09
White City Beach	30850	4627.50	66774	9128.01
Totals:	109,630	$16,444.50	248007	$35,001.06

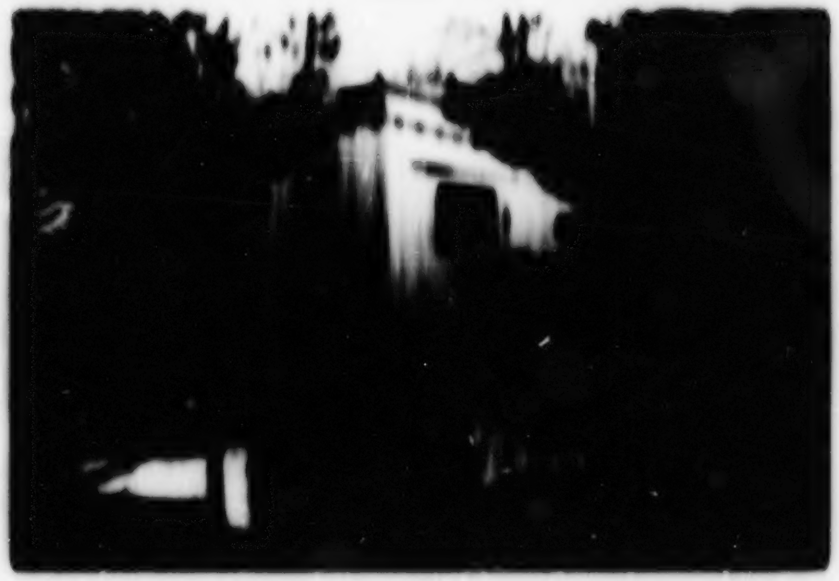

FIGURE C-1

HYPOCHLORINATION STATION

FIGURE C-7

HYPOCHLORINATION STATION

FIGURE C-3

A HOLDING TANK FOR 15% SODIUM HYPOCHLORITE WITH
A ROTO-DIP PROPORTIONATING FEEDER

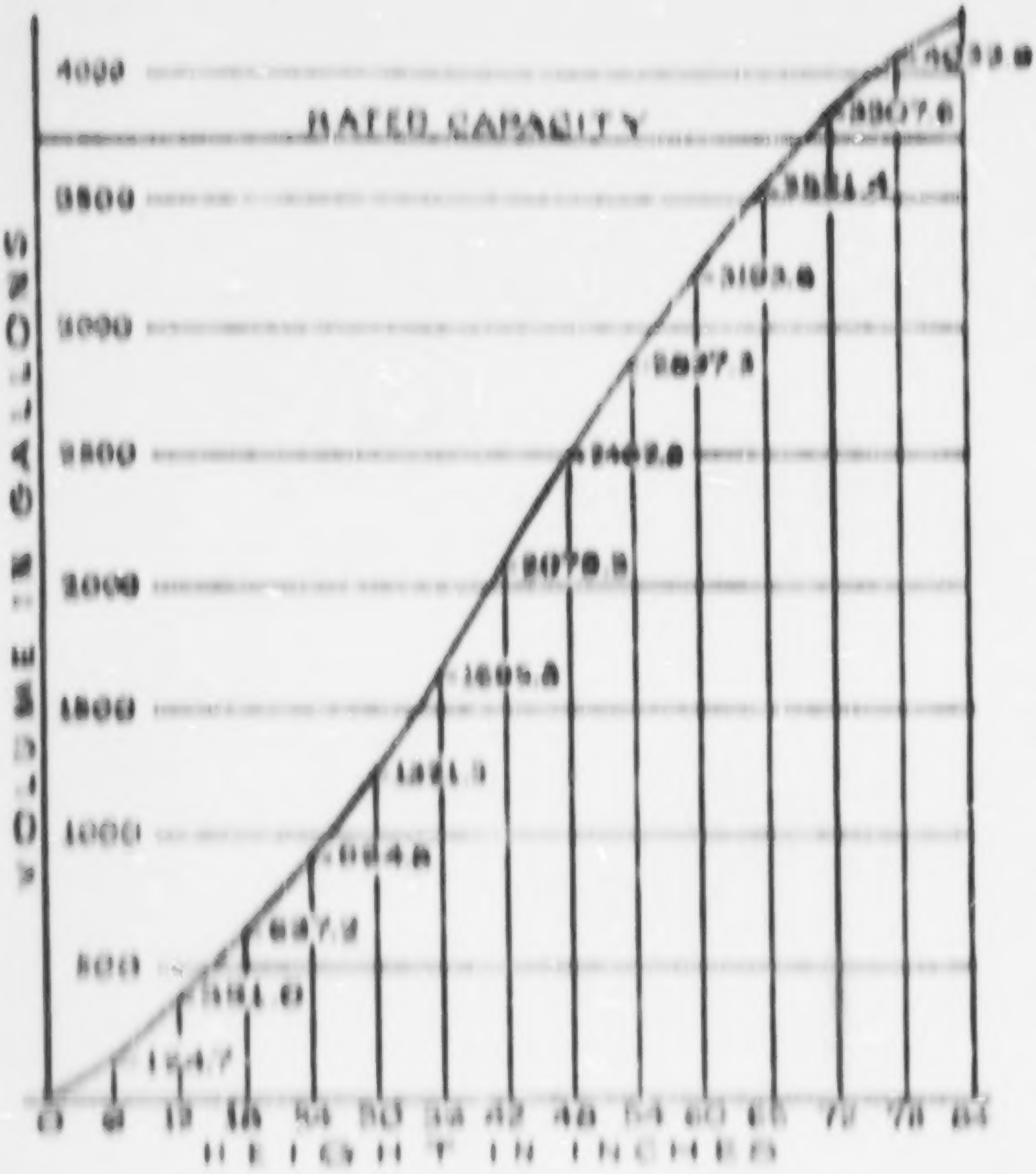

Figure C-1.

Volume curve for 3700 gallon
rated capacity chlorine tank

APPENDIX D

DYE STUDY OF DUGWAY BROOK

The flow characteristics of Dugway Brook and its dispersion in the lake were determined by a dye study. Two parameters were measured: 1) time of travel, and 2) dilution ratios of the stream discharged by lake water. Both parameters were measured over the distance between the hypochlorination station and points out into the lake. Two fluorescent dyes were employed in determining the flow travel time and dilution pattern: Rhodamine BA and Fluorescein. Fluorescein was used for travel time measurements because it can be measured with greater sensitivity than Rhodamine BA. Fluorescein has a sporadic half life, however, which makes it somewhat unsuitable for extended time studies, and Rhodamine BA dye was used for measurement of the stream-lake dilution pattern.

Fifty grams of fluorescein were introduced into the stream at the hypochlorination facility. Samples were taken from a boat at five minute intervals for the first hour of downstream travel time and at one hour intervals thereafter. Using the visible portion of the dye as an index of the approximate location of maximum concentration, samples were taken at 100 foot intervals for 400 feet on either side of the visible portion. The average elapsed time of travel was computed on the basis of peak dye concentration. Fluorescein was immediately measured using a Turner Model 111 Fluorometer carried on board the sampling vessel.

Table 1 summarizes the data taken during dry weather flow on the time travel down the stream. The data show a rapid rate of travel time from the hypochlorination facility to 100 feet downstream, and then a decreased but nearly steady rate of flow at approximately 100 feet per hour from that point to the mouth. The volume of flow had only a minor effect upon the overall rate of flow. When the volume of discharge was 1.3 mgd (dry weather flow), the average rate of flow from the downstream 100 foot mark to the mouth was 100 feet per hour. During measurements when the volume of discharge was twice the amount of dry weather flow (2.6 mgd) the average rate of travel was only 110 feet per hour. The low flood plain allows a greater volume of water to flow downstream without appreciably decreasing the time of travel. Under the stated conditions, the average time of flow from the hypochlorination facility to the mouth was sixteen hours.

One pound of dye was introduced at the mouth of the stream for measuring the rate of travel of Dugway Brook water through the lake. Samples in the lake were taken both east and west of the stream mouth at 100 foot intervals, and also directly out into the lake at 25 foot intervals for 100 feet.

Table D-1 summarizes the data for the time of travel of Dugway stream water along the lakefront. The flow was approximately 115 feet per hour, somewhat faster than the rate of flow down the stream. The total time of travel from the hypochlorination facility to 1100 feet east of the stream mouth was 21 hours. Further determinations showed that under conditions of a strong (over 10 knots) easterly wind, the stream water was moved west along the lakefront at approximately the same rate.

The second phase of the dye study, determination of dilution ratios of Dugway Brook with lake water, was done under the same weather conditions as the time of flow determinations. For determining the dispersion pattern of the stream water in the lake, Rhodamine BA dye was fed continuously into the stream at the hypochlorination facility for 25 hours. One hundred gallons of 1.0% Rhodamine BA dye was fed into the stream at a rate of 4 gallons per hour. At this point, and with the volume of discharge in the stream estimated high at 1 mgd, the dye concentration within Dugway Brook would be 100 ppb, thus allowing the concentration in 5% Dugway water to be 5 ppb, a concentration which can be reliably determined by the Turner Fluorometer.

Sixteen hours after the dye began flowing, samples were taken from Dugway Brook and along the lakefront for fluorescence determinations. As previously, samples were taken at 100 foot intervals along the lakefront and at 25 foot intervals out from the lakefront for 100 feet at both surface and depths.

Table D-2 summarizes determinations of the dilution ratios of Dugway water with lake water with respect to distance and time travel, under prevailing westerly winds.

Dilutions of the stream by the lake water began approximately 2000 feet downstream of the hypochlorination station. At 2100 feet, mixing of the two waters was considerable and varied, the percent of stream water ranging from 10% to 16%. At 2200 feet the percent of stream water varied from 5 to 16%. The wide variance of the dilution ratios in the area of 2000 feet to 2200 feet downstream was caused by alternate surges of lake water into the stream.

Table D-1. DILUTION RATIOS OF [illegible] WATER WITH LAKE WATER

Distance from Hypochlorination (feet) In Stream	 In Lake	Time of Travel (hours)	Percent [illegible] Water Present (Avg. of 4 determinations)
0		0.0	100
200		0.25	100
500		0.50	100
1300		7.75	100
1800		12.25	100
2000		14	100
2100		15	23
2200		16	11
	200' West	16	2
	1000' West	[illegible]	0.1
	10' North	16.25	1
	100' North	15.5	6
	200' East	17.75	8
	300' East	20	4
	600' East	22.5	6.5
	1000' East	25	8

In the lake there was very little variance in the dilution ratio at any particular sampling point. At a point 200 feet east of the stream mouth, the range in dilution values was 1% to 10%; at 800 feet east, 1% to 2.5%, and at 1500 feet east, 1% to 1.8%). There were no differences between the concentrations of samples taken at the surface of the lake and those taken near the bottom. At any given distance from the mouth along the lakefront, the percentage of stream water present was the same at 25 feet from the lakefront as at 100 feet from the lakefront, indicating that the bypass discharge was well mixed with lake water. Two thousand feet west of the stream mouth there was no detection of dye at any time. This point was used as a control station for comparing the effects of instream chlorination on bacterial growth in the lake.

APPENDIX E

BACTERIAL AFTERGROWTH DATA

All the chemical parameters analyzed for the Reclamation of Polluted Beachwater Project were performed in accordance with Standard Methods for the Examination of Water and Wastewater, 13th edition, by the Bureau of Wastewater Quality Control Laboratory. Field measurements of chlorine residual were made using O-Tolidine and a color comparator using the Hach Chemical Company procedure.

The Lowry Method for Protein Determination

Reagents:

Reagent A:	2% Na_2CO_3 in 0.1 N NaOH
Reagent B:	0.5% $CuSO_4 \cdot 5H_2O$ in 1% sodium tartrate
Reagent C:	Mix 50 ml of reagent A with 1 ml of reagent B. Make fresh daily.
Reagent D:	1 part phenol reagent + 1 parts water (Purchased from Fischer Scientific Company)

Method:

Combine a 1.0 ml sample (10-200μg) protein with 5.0 ml of reagent C in a 18 x 150mm culture tube and mix well. Let it stand at room temperature for ten minutes. Rapidly add 0.5 ml of reagent D and mix immediately. Allow the mixture to stand at room temperature for 30 minutes. Measure in a colorimeter at 500 mu.

Table 8-1. EFFECT OF IN-STREAM RESIDUAL CHLORINE ON AFTERGROWTH AND BASE LEVELS OF BACTERIA

Treatment		Total Coli- forms /100 ml	Fecal Coli- forms /100 ml	Fecal Strep- tococci /100 ml
1.0 mg Residual (11 days)				
Station	1	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]
	9	[illegible]	[illegible]	[illegible]
	10	[illegible]	[illegible]	[illegible]
	11	[illegible]	[illegible]	[illegible]
3.0 mg Residual ([illegible] days)				
Station	1	[illegible]	[illegible]	0
	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]
	9	[illegible]	[illegible]	[illegible]
	10	[illegible]	[illegible]	[illegible]
	11	[illegible]	[illegible]	[illegible]
5.0 mg Residual ([illegible] days)				
Station	1	[illegible]	[illegible]	0
	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]
	9	[illegible]	[illegible]	[illegible]
	10	[illegible]	[illegible]	[illegible]
	11	[illegible]	[illegible]	[illegible]

Table I-1. CHEMICAL PARAMETERS OF IN-STREAM SAMPLING STATIONS OF [illegible] [illegible]

Prior to Chlorination (Pre-Cl) and After Chlorination (Cl)

	1		2		3		4		5	
	Pre-Cl	Cl	Pre-Cl	Cl	Pre-Cl	Cl	Pre-Cl	Cl	Pre-Cl	Cl
No. Samples	5	6	4	6	4	6	5	5	4	6
Alkalinity	230	245	288	240	241	245	265	208	228	99
pH	7.6	7.7	7.6	7.8	7.9	7.9	7.7	8.6	7.8	8.1
Chloride	259	156	261	232	224	152	76	73	55	30
Ammonia	1.8	1.9	0.27	1.9	0.9	1.20	3.8	0.55	0.7	0.58
Nitrates	1.27	1.26	1.35	0.74	1.42	0.86	0.75	0.50	0.80	1.43
Nitrites	0.104	0.115	0.32	0.062	0.125	0.058	0.105	0.28	0.09	0.088
Organic N	1.6	1.4	1.9	1.9	0.28	1.55	1.9	1.46	1.1	1.46
Phosphorus	1.30	1.06	1.48	0.9	1.25	0.28	2.1	0.8	1.25	0.9
BOD	28	11.6	28	6.7	12.1	7.3	7.6	6.1	6.1	3.1
COD	42	34	8.7	38	65	38	52	32	38	42
Spec. Cond.	1180	830	1110	670	780	800	400	320	400	345
Res. Evap.	718	711	715	670	538	735	275	270	280	320
Diss. Solids	682	425	701	380	511	640	230	271	234	158
Hardness	302	338	385	280	280	318	285	262	285	237
Turbidity	33	28	34	45	29	11.3	14	9.1	0.1	8.8

All values except pH and specific conductivity are given as mg/l.

Table 8-1. CHEMICAL PARAMETERS OF LAKE SAMPLING STATIONS ADJACENT TO INTAKE AREA

	Lake Stations Prior To Chlorination			Lake Stations During Chlorination		
Parameter	Ave.	Max.	Min.	Ave.	Max.	Min.
Alkalinity	107	111	98	88	98	80
pH	8.1	8.5	7.8	8.1	8.5	7.2
Chloride	42	67	22	42	48	36
Ammonia	0.41	1.1	0.4	0.44	1.1	0.1
Nitrates	0.65	1.80	0.20	0.47	1.98	0.20
Nitrites	0.033	0.06	0.001	0.027	0.191	0.016
Organic N	0.48	1.1	0.7	0.29	1.9	0.7
Phosphorus	0.17	0.3	0.07	0.40	1.8	0.1
BOD	4.1	8.8	0.3	1.07	9.2	0.8
COD	28	41	4	21.5	38.2	2.8
Spec. Cond.	361	482	311	326	338	271
Hardness	138	180	128	188	182	128
Diss. Matter	198	288	185	252	248	208
Res. Evap.	248	333	206	238	382	201
Protein N	1.1	10	0	1.07	12	0
Diss. O_2	7.6	8.4	6.8	7.4	8.2	6.3

All values except pH and specific conductivity are given as mg/l.

Table 3-4. CHEMICAL ANALYSES FOR BENCH SCALE BACTERIAL ACTIVATED SAMPLES

Parameter	Initial	Activated 24 hr.	48 hr.	72 hr.	Chlorinated	Chlorinated 24 hr.	48 hr.	72 hr.
NH_3	7.8	6.7	6.3	5.2	7.8	5.6	5.8	7.4
NO_3	0.07	0.42	0.28	0.22	0.40	0.23	0.50	0.32
NO_2	0.010	0.222	0.02	0.02	0.065	0.067	0.078	0.248
Org. N	4.5	1.4	1.1	2.6	4.3	2.3	2.5	8.9
TS	17.141	6.602	8.90	12.22	15.065	8.737	6.974	24.338
BOD	14.7	—	—	—	14.7	—	—	—
COD	7.9	38	40	111	47	33	94	38
SS	16	10	26	11	26	11	20	8
P	0.9	1.8	—	1.2	1.3	1.0	1.1	1.1
Alk.	164	166	166	148	166	111	625	148
Diss. O	107	111	40	114	139	139	127	175
TOC	—	47	—	—	—	72	—	—
Turb.	23	29	32	23	82	71	74	62

Table B-1

	Blended				Blended then Chlorinated			
Parameter	Initial	24 hr.	48 hr.	72 hr.	Initial	24 hr.	48 hr.	72 hr.
pH	7.3	6.7	6.9	7.0	6.9	7.3	6.6	6.7
NH_3	0.61	0.14	0.27	0.20	0.42	0.27	0.67	0.27
NO_2	0.10	0.09	0.08	0.023	0.075	0.059	0.232	0.064
Org. N	1.5	1.6	1.7	6.8	4.8	2.8	2.8	4.1
TN	27.14	0.51	0.11	24.133	21.695	0.319	0.362	21.238
BOD	23	—	—	—	22	—	—	—
COD	46	46	56	71	46	33	68	73
SS	29	29	28	32	28	8	22	22
P	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
Alk.	159	186	661	181	665	179	659	654
Diss. S	127	129	129	146	156	188	141	659
TOC	—	61	—	—	—	60	—	—
Turb.	71	59	78	72	86	80	76	62

Table 5-4

Parameter	Initial	Chlorinated Time Elapsed 24 hr.	48 hr.	72 hr.
NH_3	7.1	1.6	1.5	6.3
NO_3	0.47	0.23	0.12	0.18
NO_2	0.073	0.068	0.128	0.285
ON	4.7	1.8	1.9	4.6
TN	12.343	3.698	3.648	11.365
BOD	28	—	—	—
COD	24	49	56	117
SS	10	18	28	6
P	10	1.1	—	1.1
Alk.	162	116	417	432
Cond. *	150	133	150	415
TOC	—	48	—	—
Temp.	42	42	44	33

PROTEIN CONTENT VERSUS BACTERIAL GROWTH

In order to determine the effects of chlorination upon the proteinaceous organic content and subsequent growth of bacteria, the following experiment was conducted.

A two liter sample from [illegible] Brook was filtered through a 0.45 µ membrane filter. Thus, all of the suspended matter was removed from the sample, and from consideration as a variable in bacterial growth. Also, all but the very smallest organism were removed, thus eliminating competition among bacterial species for available nutrients.

The filtrate was divided into ten 200 ml volumes. The filtered samples were treated with sodium hypochlorite to produce 0, 1.0, 3.0, 6.0, and 12.0 mg/l residual chlorine concentrations with two samples at each level.

After 30 minutes contact time, the residual chlorine was neutralized with filter sterilized sodium thiosulfate. The samples were innoculated with two indicator bacteria species, *A. aerogenes* and *E. coli*. The samples then were incubated at [illegible] for 72 hours. Analysis of bacterial density was conducted at 24 hour increments. The amount of available reactive proteinaceous organic nitrogen was determined also.

Figure 1 describes the growth of the bacterial species at peak growth versus the residual chlorine dosage. The samples containing the *A. aerogenes* all showed a steady increase in bacterial density daily to 72 hours except for the sample treated with 12 mg/l residual chlorine. The 12 mg/l residual chlorine sample decreased from the initial level of [illegible]/100 ml to [illegible]/100 ml, but increased after that by a log unit daily.

Of the samples innoculated with *E. coli*, two showed a decrease in density after 24 hours. The six mg/l residual sample decreased from [illegible]/100 ml to [illegible]/100 ml, and the 12 mg/l residual chlorine sample decreased from [illegible]/100 ml to [illegible]/100 ml.

Table [illegible] summarizes the results of the analysis for available reactive proteinaceous organic nitrogen, as determined by the Lowry Method.

As can be seen in the above table, the amount of reactive proteinaceous organic nitrogen increases rapidly as the

Table E-7. REACTIVE PROTEIN NITROGEN (mg/l)

Incubation time	Residual chlorine dosage (mg/l) 0	1	3	6	12
initial	14	36	61	88	146
48 hr.	12	16	22	31	37
72 hr.	11	10	12	16	18

chlorine dosage is initially applied. The proteinaceous material decreased as the bacterial population increased. The resulting protein concentration after 72 hours was lower for those samples which displayed the greatest peak growth of bacteria. In view of this evidence it was hypothesized that the chlorination cleaves proteins into smaller and more readily useable forms or in some manner alters the form of the proteins to make them more reactive to the Lowry test and more readily useable by bacteria. Further investigation into the available protein relationship with bacterial aftergrowth has not been attempted at this time.

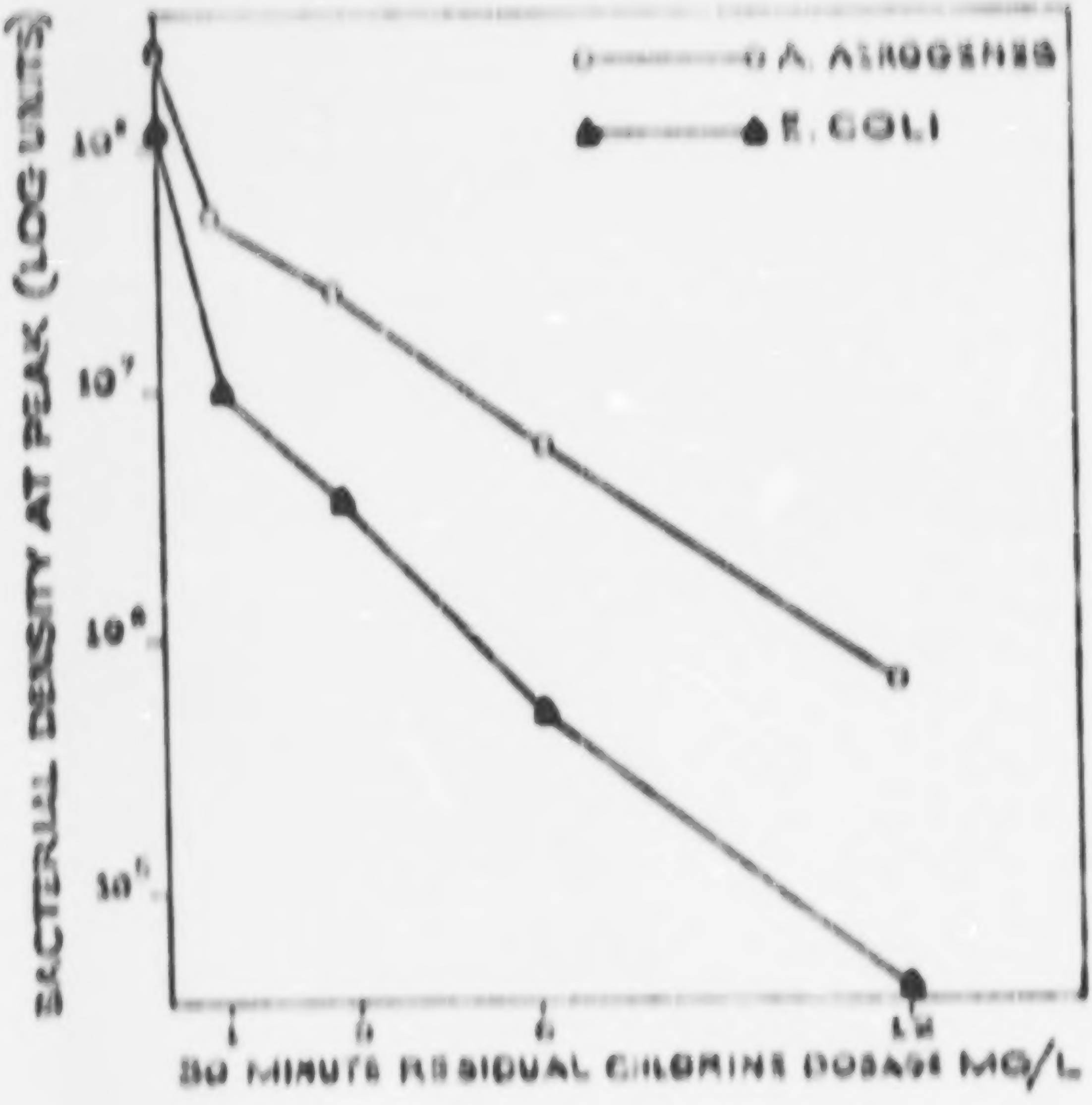

FIGURE E-1

Peak growth of bacteria in chlorinated wastewater

APPENDIX F

SEWER FLUSHING DATA

Table F-1

	West 102nd Street	West 103rd Street
Combined Sewer Size	12"	12"
Test Section Length	300'	140'
Test Section Slope	0.40%	0.10%
Buildings in drainage area	24 double houses	17 double houses
Treatment Center serving this area	Westerly	Westerly
Interceptor sewer serving this area	Westerly via Lake Avenue, Baltic Road	Westerly via Lake Avenue, Baltic Road
Major Overflows	Edgewater Park	Edgewater Park

Table F-2

	(at n=.013)	(at n=.015)	(at n=.013)	(at n=.015)
Discharge to cause 2 fps	0.13 cfs	0.10	0.20	0.18
Depth of flow for 2 fps	1.3"	1.2"	2.1"	1.8"
Discharge to cause 6 fps	1.3 cfs	0.8	2.6 (1.7)*	1.6
Depth of flow for 6 fps	1.9"	1.6"	6" (1.7)*	6"

Table F-3. PHASE I - WEST 103rd STREET SEWER FLUSHING PROGRAM
Velocities of 2 ft/sec and 3.7 ft/sec

Number	Date	Flow Rate	Suspended Solids (mg/l) F	Suspended Solids (mg/l) I	C.O.D. (mg/l) F	C.O.D. (mg/l) I
1	10/9/70	2 ft/sec	812	142	810	1073
2	"	"	636	94	553	73
3	"	"	152	28	88	28
4	"	"	66	27	85	12
5	"	"	86	12	95	8
6	"	"	51	26	28	28
7	"	"	26	13	33	18
8	"	"	18	13	203	57
9	"	"	20	8	88	28
10	"	"	82	22	38	20
1	10/9/70	3 ft/sec	88	18	61	12
2	"	"	97	18	73	12
3	"	"	38	81	37	18
4	"	"	31	19	33	8
5	"	"	28	10	28	8
6	"	"	21	18	18	110
7	"	"	23	18	20	12
8	"	"	82	21	118	103
9	"	"	51	85	110	28
10	"	"	29	68	57	188

Samples are in order of collection. Each sample was taken one minute apart beginning with the first wave of the flush.

Table F-4. SEWER FLUSHING PROGRAM
PRELIMINARY TESTING PERIOD 9-21-70 TO 9-28-70
(Suspended Solids and COD in mg/liter)
West 102nd Street

Date	Time	Man Hole	Water Depth C	Water Depth B	Suspended Solids C	Suspended Solids B	C.O.D. C	C.O.D. B	Daily Rainfall
9/21/70	9:00 AM	C	3.5"		78		810		Trace
"	9:00 AM	B		1.5"		122		203	
"	12:30 PM	C	3.375"		58		281		
"	12:30 PM	B		1.25"		90		301	
9/22/70	9:00 AM	C	3.5"		66		88		0.20"
"	9:00 AM	B		1"		100		58	
"	12:30 PM	C	3.5"		76		188		
"	12:30 PM	B		1"		88		232	0.18"
9/23/70	9:00 AM	C	3.25"	122			160		
"	9:00 AM	B		1"		220		130	
"	12:30 PM	C	6"		72		571		
"	12:30 PM	B		1"		128		811	
9/24/70	9:00 AM	C	1"		26		281		0.25"
"	12:30 PM	C	3.25"		128		106		
9/25/70	9:00 AM	C	3.25"		188		168		0.02"
"	12:30 PM	C	3.25"		60		311		
9/26/70	9:00 AM	C	6.0"		118		870		0.30"
"	12:30 PM	C	6.75"		112		887		
9/27/70	9:00 AM	C	3.75"		76		128		0.18"
"	12:30 PM	C	3.5"		98		870		
9/28/70	9:00 AM	C	6"		62		330		0.21"
"	12:30 PM	C	3.75"		316		570		

	Water Depth C	Water Depth B	Suspended Solids C	Suspended Solids B	C.O.D. C	C.O.D. B
MAXIMUM	6.75	1.5"	316	220	571	330
MINIMUM	1"	1"	26	88	128	232
AVERAGE	3.81	1.12"	98.9	133.7	303.9	153.8

Table F-1. SEWER FLUSHING PROGRAM
PRELIMINARY TESTING PERIOD 9-21-70 TO 9-28-70
(Suspended Solids and COD mg/liter)
West 103rd Street

Date	Time	Man Hole	Water Depth F	Water Depth L	Suspended Solids F	Suspended Solids L	C.O.D. F	C.O.D. L	Daily Rainfall
9/21/70	9:00 AM	F	7.75"		48		245		Trace
"	9:00 AM	L		1.25"		174		277	
"	12:30 PM	F	NR		148		454		
"	12:30 PM	L		1.50"		50		96	
9/22/70	9:00 AM	F	7.5"		20		NR		0.20"
"	9:00 AM	L		NR		288		NR	
"	12:30 PM	F	7.375"		46		112		
"	12:30 PM	L		2.125"		102		216	
9/23/70	9:00 AM	F	7.75"		338		490		0.18"
"	9:00 AM	L		1.25"		118		217	
"	12:30 PM	F	7.375"		48		206		
"	12:30 PM	L		1.375"		336		711	
9/24/70	9:00 AM	F	NR		26		241		0.23"
"	12:30 PM	F	8.50"		120		427		
9/25/70	9:00 AM	F	8.50"		88		212		0.02"
"	12:30 PM	F	8.75"		124		408		
9/26/70	9:00 AM	F	8.25"		64		341		0.30"
"	12:30 PM	F	8.25"		28		274		
9/27/70	9:00 AM	F	NR		8		128		0.16"
"	12:30 PM	F	8.75"		26		146		
9/28/70	9:00 AM	F	NR		150		185		0.21"
"	12:30 PM	F	8.50"		66		240		

	F	L	F	L	F	L
MAXIMUM	8.75"	2.125"	338	336	490	711
MINIMUM	7.375"	1.25"	8	50	112	96
AVERAGE	8.1"	1.5"	85.5	178	280.7	303.4

Table F-6. PHASE I - WEST 102nd STREET SEWER FLUSHING PROGRAM
Flow Velocities of 2 ft/sec and 3.7 ft/sec

Number	Date	Flow Rate	Suspended Solids C	Suspended Solids R	C.O.D. C	C.O.D. R
1	10/20/70	2 ft/sec	1888	988	1120	1107
2	"	"	920	576	720	204
3	"	"	288	188	80	40
4	"	"	184	108	56	8.0
5	"	"	68	62	64	28
6	"	"	68	68	80	28
7	"	"	84	88	88	5.0
8	"	"	60	60	80	8.0
9	"	"	18	28	80	12.0
10	"	"	106	72	152	788
1	10/20/70	3 ft/sec	88	128	16	80
2	"	"	266	108	180	88
3	"	"	102	78	32	58
4	"	"	102	78	28	18
5	"	"	78	62	28	28
6	"	"	68	80	32	80
7	"	"	54	58	20	12
8	"	"	58	86	32	5.0
9	"	"	82	88	5.0	5.0
10	"	"	56	28	12.0	5.0

Samples are in order of collection. Each sample was taken at one minute intervals beginning with the first wave of the flush.

Table F-7. WEST 103rd STREET SECOND TEST
3.7 Feet Per Second
Suspended Solids (mg/liter)

Number	Date	Suspended Solids 1	2	Manhole
1	11/19/70	1264	2064	
2	"	286	102	
3	"	138	68	
4	"	108	44	
5	"	64	36	
6	"	26	40	
7	"	66	50	
8	"	60	30	
9	"	30	40	
10	"	34	36	

Whereas the first test of Phase I consisted of taking samples at 2 ft/sec then 3.7 ft/sec immediately thereafter, the second test was conducted using a velocity of 3.7 ft/sec only.

Table F-8. PHASE II - WEST 102nd STREET SEWER FLUSHING PROGRAM
Suspended Solids and C.O.D. - mg/liter

		Suspended Solids		C.O.D.	
Number	Date	C	B	C	B
1	11/9/70	1434	1776	692	319
2	"	988	336	262	169
3	"	260	98	77	223
4	"	180	166	46	23
5	"	170	270	42	23
6	"	150	204	46	31
7	"	138	176	42	73
8	"	144	236	27	63
9	"	136	33	83	30
10	"	138	144	88	13

Samples are in order of collection. Each sample was taken at 30 second intervals beginning with the first wave of the flush.

Table F-9. PHASE II - WEST 10th STREET SEWER FLUSHING PROGRAM
(Suspended Solids and COD mg/liter)

		Suspended Solids		C.O.D.	
Number	Date	2	3	2	3
1	11/9/79	886	2746	865	2381
2	"	1230	158	3038	388
3	"	2600	132	2323	113
4	"	1132	80	1363	92
5	"	652	102	392	108
6	"	228	60	333	82
7	"	286	180	130	112
8	"	736	1202	318	180
9	"	738	170	333	280
10	"	138	92	92	88

Samples are in order of collection. Each sample was taken at 30 second intervals beginning with the first wave of the flush.

NOTE: Flushing began at manhole 1 sample collection thus took place first at 2, then 3.

Table F-10. PHASE III - WEST 103rd STREET SEWER FLUSHING PROGRAM
(Suspended Solids and COD - mg/liter)

Number	Date Collection	Suspended Solids Gutter	7	C.O.D. Gutter	7
1	10/27/70	252	136	369	40
2	"	258	6	365	20
3	"	250	118	362	24
4	"	476	34	765	4.0
5	"	1808	32	1508	8.0
6	"	880	24	271	8.0
7	"	528	20	304	6.0
8	"	134	14	120	16
9	"	262	28	105	12
10	"	304	20	145	6.0

Samples are in order of collection. Each sample was taken at 30 second intervals beginning with the first wave of the flush.

NOTE: Flushing began above catch basin. Sample collection thus took place first at catch basin and then at manhole 7.

Table 9-11. CALCULATED FLOW DATA - WEST 240th STREET

$v = 1.318CR^{0.63}S^{0.54}$
v = velocity in feet per second
R = hydraulic radius in feet
S = slope of energy gradient in
feet per second

Velocity	Cubic Ft/Sec	Gallons Daily	Gallons/ Minute	Gallons/ Second	Depth
0.25 ft/sec (normal)	0.025	15,000	10.7	0.02	1.62"
2 ft/sec	0.28	424,000	294.9	4.92	2.2"
3.7 ft/sec	1.4	784,000	544.7	9.1	6"

Depth with weir

3.7 = highest velocity

Table F-11. FLUSH DATA - WEST 162nd STREET, WEST 163rd STREET

Phase	Location	Period of Flush	Ft/Sec	Gallons Flush Period	Pounds/Flush Suspended Solids	Pounds/Flush COD	Pounds/Gallon Suspended	Pounds/Gallon COD
I	C @ 162	20 Minutes	2	22,344.7	562.0	225.2	0.027	.01
I	D @ 162	20 Minutes	2	22,344.7	292.6	248.8	0.020	.013
I	C @ 162	20 Minutes	2.7	29,945.99	174.4	93.40	0.007	.003
I	D @ 162	20 Minutes	2.7	29,945.99	125.8	40.24	0.006	.002
I	F @ 163	20 Minutes	2	22,344.7	138.3	117.92	.012	.014
I	E @ 163	20 Minutes	2	22,344.7	54.2	124.37	.005	.011
I	F @ 163	20 Minutes	2.7	29,945.99	93.9	89.9	.003	.0045
I	E @ 163	20 Minutes	2.7	29,945.99	42.1	84.10	.002	.0024
I	F @ 163	20 Minutes	2.7	29,945.99	330.3	NR	.020	NR
I	E @ 163	20 Minutes	2.7	29,945.99	385.3	NR	.023	NR
II	C @ 162	5 Minutes	2.7	5,473.2	329.7	111.0	.033	.011
II	F @ 163	20 Minutes	2.7	29,945.99	71.3	25.3	.004	.001
II	F @ 163	5 Minutes	2.7	5,473.2	831.2	789.6	0.07	.074
II	E @ 163	5 Minutes	2.7	5,473.2	410.3	362.8	0.04	.03
III	F @ 177	5 Minutes	2.7	29,945.99	288.7	34.2	.051	.0089
III	Gutter @ 163	20 Minutes	2.7	29,945.99	728.7	731.5	.038	.036

NR - Not Recorded

Table F-13. CHARACTERISTICS OF COMBINED SEWER OVERFLOW

Parameter	Concentration
C.O.D.	218
B.O.D.	38
Suspended Solids	120
Total Solids	381
Phosphorus (as P)	8
Total Nitrogen	12

Table F-14. POLLUTION LOADS FROM COMBINED SEWER OVERFLOW

Parameter	Total Annual Discharge (lbs)
C.O.D.	21,000,000
B.O.D.	7,110,000
Suspended Solids	11,310,000
Total Solids	31,880,000
Phosphorus (as P)	1,098,000
Total Nitrogen (as N)	1,808,000

APPENDIX C

VELOCITY MEASUREMENT IN SEWERS

The salt concentration was used to measure the velocity (V) and the depth of flow was measured to compute the area (A). Discharge (Q) was then computed by Q=AV. Sewers to be investigated had a section selected with a constant slope and size and without any branches coming into the length of the pipe between the two manholes chosen, and analysis of data will be discussed in the following section.

EQUIPMENT USED

The equipment used to measure velocity by the salt-concentration method can be purchased at the local hardware store. A nine volt dry cell battery, a milliammeter, enough wire for the deepest manhole, stopwatch, and tape constitute the measuring apparatus. A weight was required to hold the wire and electrodes at the proper depth in the flow, and an iron bar was laid across the manhole rim to secure the rope and weight. A portable tape recorder was used instead of writing the time and ammeter reading. The recorder is not necessary, but it greatly reduces the chance of missing a reading because of illegible writing or not being able to write fast enough to record all of the readings and times. One of the weights used was 1½" pipe 18" long, filled with lead. One end was bullet shaped, and a vane was fastened to the other end to orient it parallel to the flow. The other weight was a standard 15 pound weight used for stream gage measurements. The ammeter had a range of 0-50 milliamps and was purchased at a surplus store. The equipment is shown in Figure C-1.

PROCEDURE

A survey was made to determine the relative elevation of manhole rims and inverts, the distance between manholes, and to verify the size and slopes of the sewer. Once the rim to invert distance was measured, the depth of flows were defined by measuring the rim to water surface distance and subtracting from the rim to invert distance. Manhole rims were checked for being level with a 3' carpenters level, and a temporary bench mark was established by notching the rim for consistency in measuring depths.

The distance from the rim to the water surface was measured at both upstream and downstream manholes just prior to and at the end of each test.

The weight with the wire and electrodes attached was lowered into the water so the electrodes were at the 0.6 depth of flow. It was tied at this depth to the bar across the manhole rim. This is shown in Figure 1. The salt was dumped at the upstream manhole from a five gallon pail which was lowered just above the water surface. The pail was tipped by a second rope fastened through a hole in the bottom, and the stopwatch was started by the person at the downstream manhole when he saw the bucket being tipped. Granulated table salt was used and was dumped dry. The amount of salt used was approximately one pound per estimated one CFS of flow.

A wooden float was placed on top of the salt, and consequently was dumped at the same time. The time of arrival of the float was recorded and later compared with the arrival of the salt. The person at the downstream manhole started reading the ammeter and stopwatch to define the background. As soon as the salt concentration started to increase, the frequency of readings was increased. The average test required a reading every few seconds to define the concentration graph. This was continued until the ammeter readings were back close to the background values. Three tests were usually made at each depth of flow. The total time from arrival to finish for a crew of three men to set up and do the three tests was about 60 minutes.

ANALYSIS OF DATA

The ammeter readings were plotted with respect to the time, and a typical concentration graph is shown in Figure 2. The centroid was then determined by estimating its location and planimetering the area on both sides of the assumed centroid. The position of the assumed centroid was then adjusted until the area under the graph was equal for both sides. This gives the average time for the salt mass to travel the distance between the two manholes and is used to compute the average velocity.

Since the depth of flow and the size of the pipe is known, the area, wetted perimeter and hydraulic radius can be computed or taken from tables. For this study a simple computer program was used to produce a table giving these values plus the hydraulic radius to the two-thirds power for every common pipe size between 6" and 180" for about every 1% increment in flow depth from zero to full.

The discharge was plotted against the depth of flow to develop a rating curve which could be used to convert depths of flow, as measured continuously by a stage recorder, to discharge. Figure [illegible] shows a typical rating curve. The continuous recorder over a long period of time will give a good average of minimum, maximum and average daily flows.

The float which was dumped with the salt gave the surface velocity. The surface velocity was always greater than the average velocity and always arrived at the beginning of the increase in salt concentrations as is shown in Figure [illegible]. A ratio of average velocity to surface velocity was computed and plotted against the ratio of depth to diameter for all tests. Hopefully, a ratio could be found that would give accurate average velocities if the surface velocity was measured. The ratio varied from [illegible] to [illegible] with an average [illegible], which can be used to approximate values of average velocity, but it should not be used for any precise need.

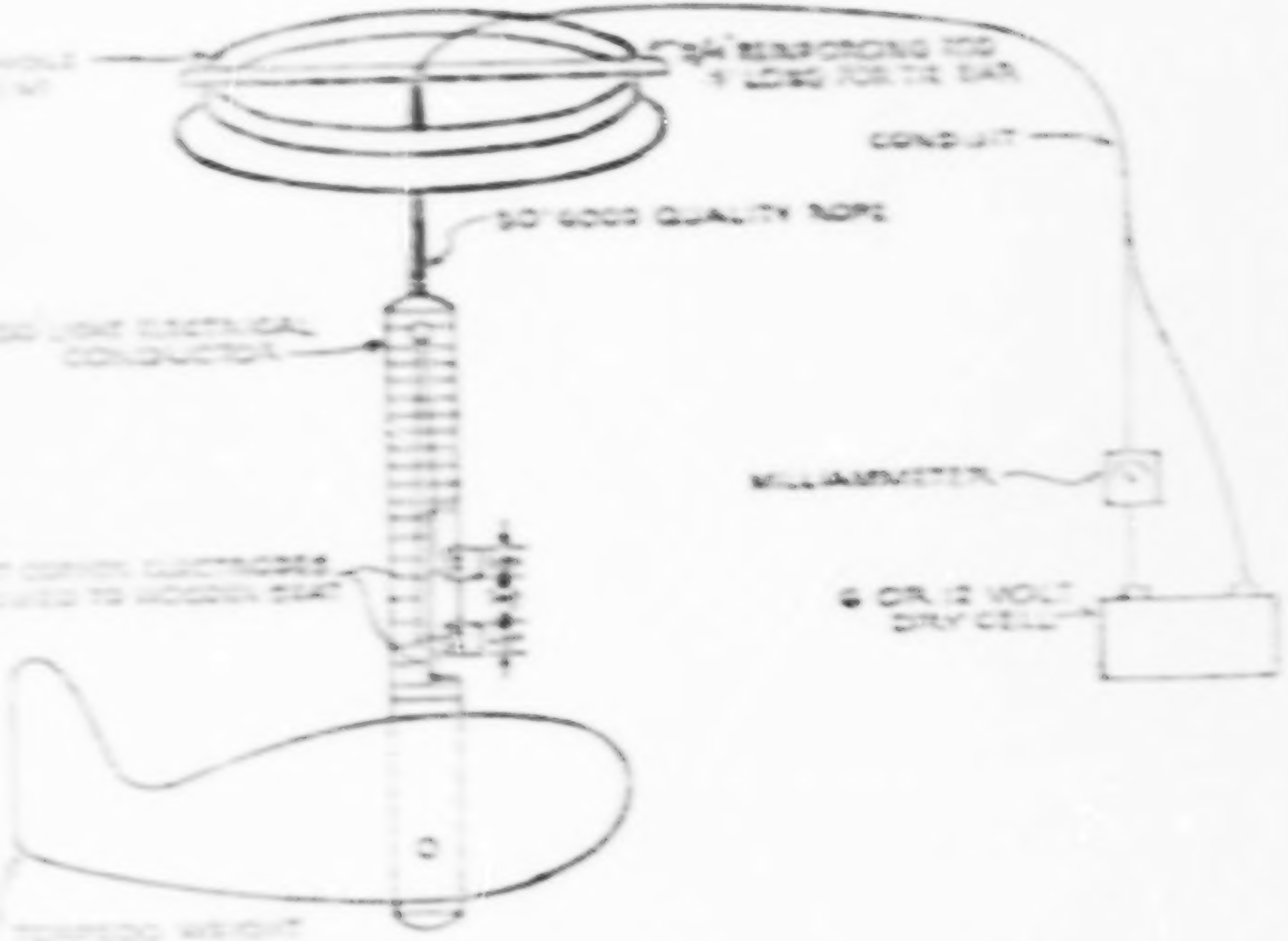
MANHOLE RING
3/4" REINFORCING ROD 4' LONG FOR TIE BAR
CONDUIT
50' GOOD QUALITY ROPE
50' LIGHT ELECTRICAL CONDUCTOR
MILLIAMMETER
6 OR 12 VOLT DRY CELL
SHEET COPPER ELECTRODES ATTACHED TO WOODEN SEAT
TORPEDO WEIGHT

Figure D-1.

Velocity measurement apparatus

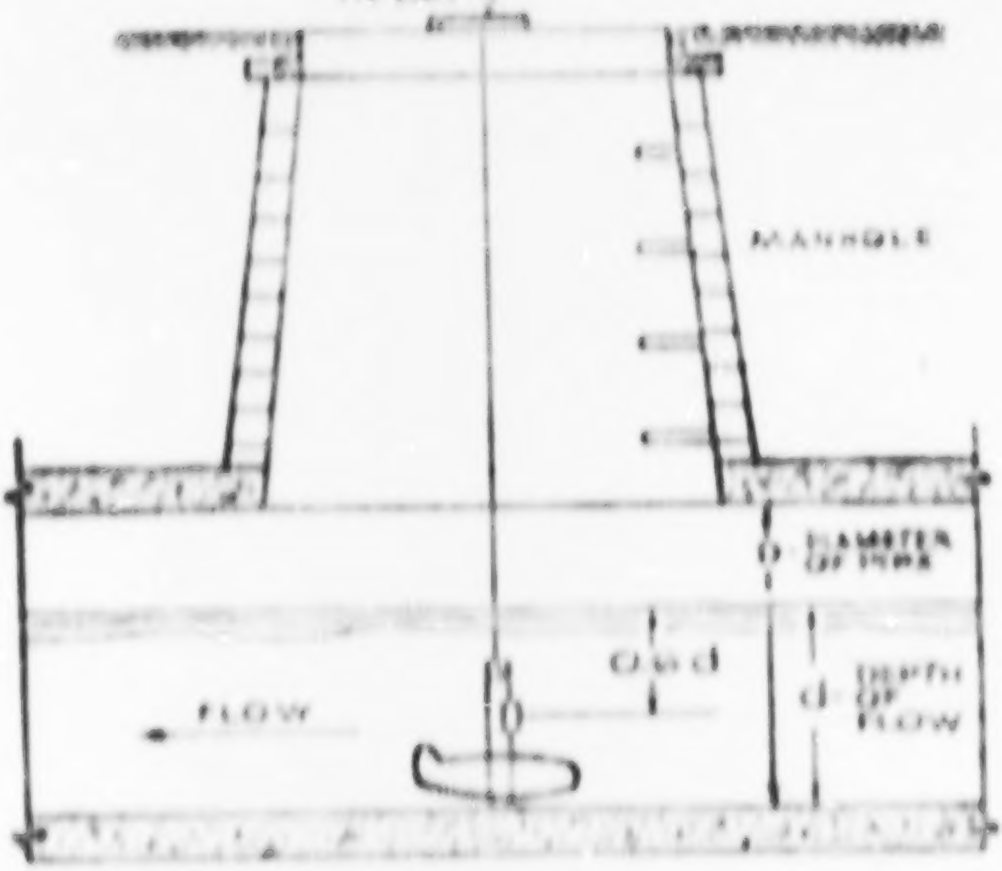
TIE BAR
MANHOLE
D = DIAMETER OF PIPE
0.6 d
d = DEPTH OF FLOW
FLOW

Figure 5.1.

Placement of electrodes in flow

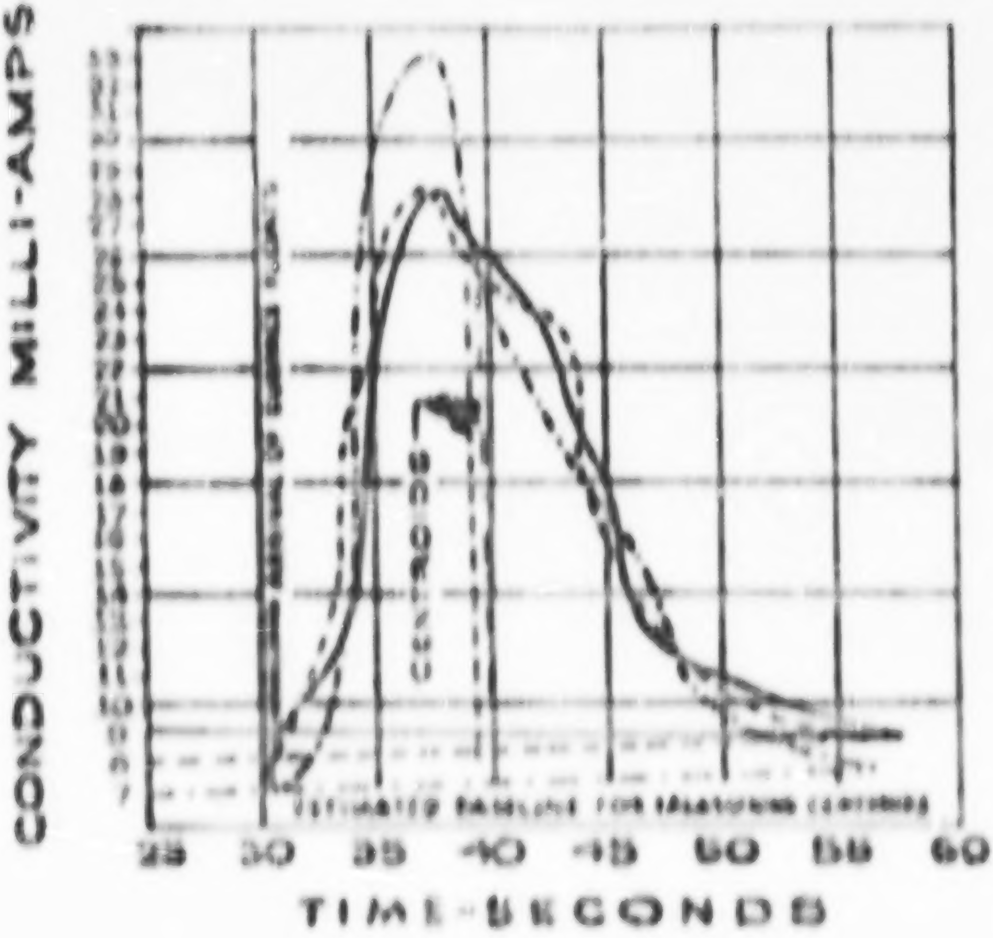

Figure [illegible]

Typical concentration graph

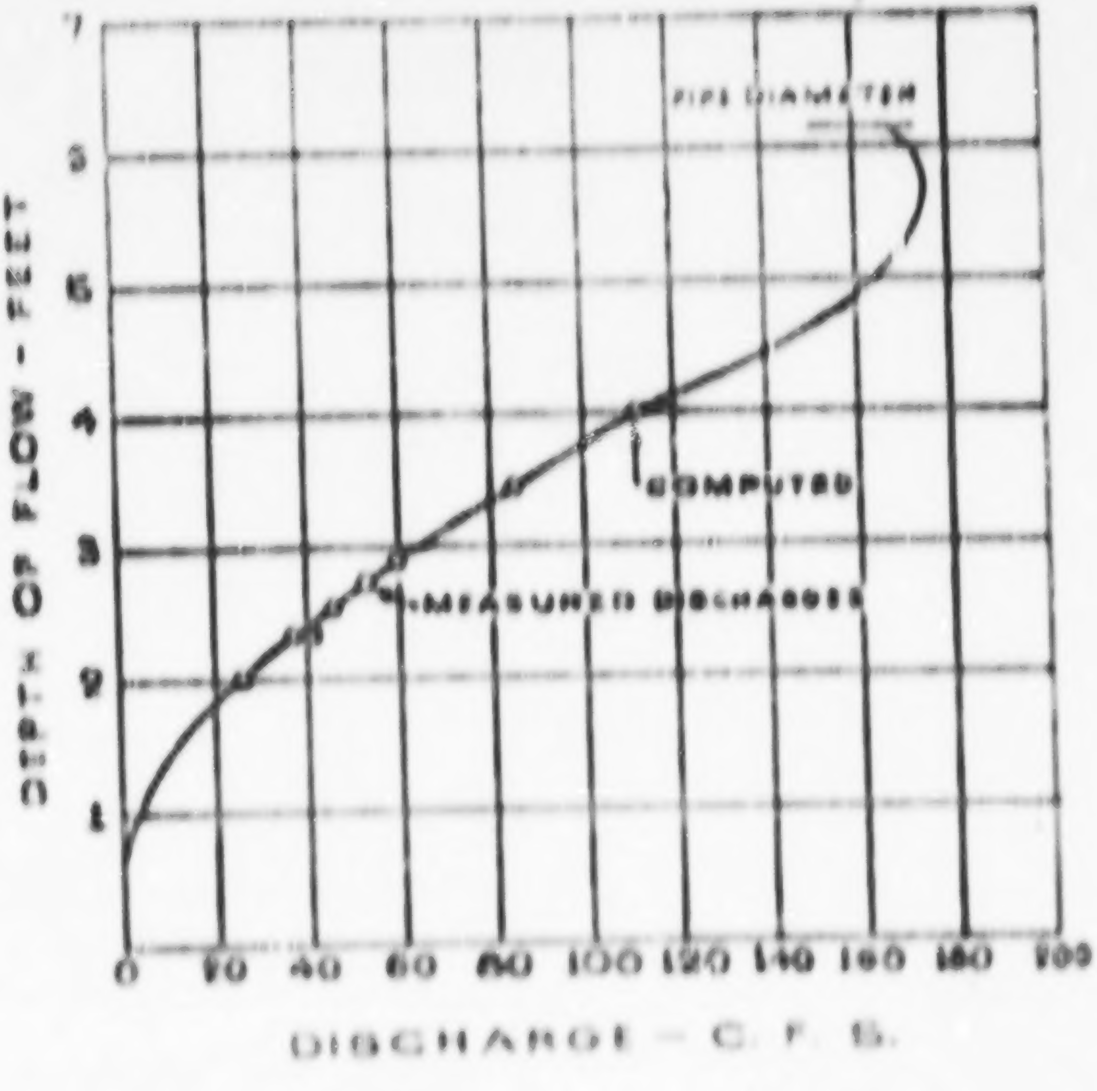

Figure C-1.

Rating curve

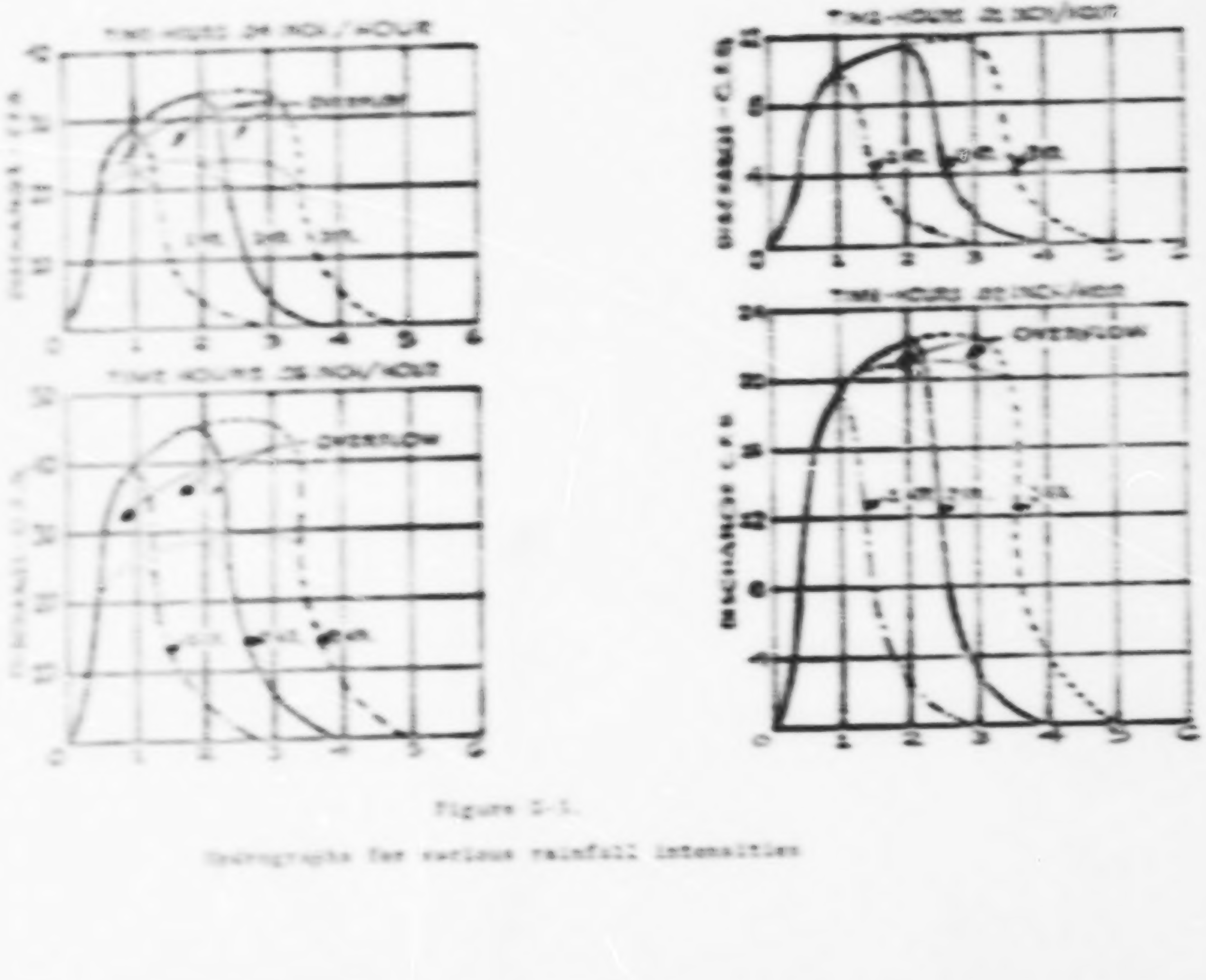

Figure 2-1.

Hydrographs for various rainfall intensities

APPENDIX H

SPECIFICATIONS: BEACH CLEANING EQUIPMENT

One Allis-Chalmers Model One-Ninety Beachmaster Tractor and Sanitizer which includes as one unit:

1. One 190 Beachmaster Tractor with a diesel engine, 12-Volt System, Lighting, 3 Point Hitch, PTO, Scarifier, and all necessary equipment to operate the pull-type sanitizer.

2. One pull-type Beach Sanitizer with Lifting Reel, Paddles, Gear Boxes, Hydraulic Trash Bucket (1½ Cu. Yd.), 8' Rear Sand Float, Coarse Conveyor Belt, Conveyor Brushes, and all necessary equipment for proper operation and hook-up to the prime mover.

3. Front Mounted Loader Attachment for the prime mover.

4. Also available in lieu of the standard coarse screen conveyor belt (1¼ x 1½ x 7 gauge), a fine conveyor screen can be offered (1" x 1" x 7 gauge).

Table H-1. BEACH SANITIZER OPERATION DATA
(Two Season Total)

Number of times beach cleaned at Edgewater		72
Number of times beach cleaned at White City.....		44
Total for both beaches..........................		116
Tonnage removed at Edgewater....................		86
Tonnage removed at White City...................		38
Total tonnage for both beaches..................		124
Man hours used (inc. transp. between beaches).....		1250
Average rate per hour.............................	$	1.86
Total man power costs.............................	$	2,325.00
360 gals. Diesel Fuel @ .30.......................	$	108.00
Total operating costs.............................	$	2,433.00
Average cost per ton removed (less amortization)	$	19.62
Average tonnage removed per cleaning..............		1.1

FIGURE D-1

VIEW OF THE SEPARATOR SECTION
OF THE BEACH SANITIZER

Debris is disengaged from the sand with the aid of the action of the brushes. A variety of screens may be used to pass larger or smaller objects.

FIGURE [illegible]

[illegible] BEACH AFTER "SANITIZING"

FIGURE [illegible]

BEACH SANITIZER WITH PRIME MOVER

APPENDIX I

COST OF THE PROJECT

The costs listed in the following tables are the costs actually incurred during the period of this demonstration project from 1964 to 1970. They do not reflect any current increase in cost of commodities and services at the 1973 prevailing rates.

Table I-1. SUMMARY OF COSTS

Cost of Project	$ [illegible]
Labor	[illegible]
Material	[illegible]
Total Cost	$ [illegible]
Federal Participation	$ [illegible]
City of Cleveland Participation	[illegible]
Total	$ [illegible]

Table I-2. REPORT OF COST INCURRED

Section II Pollution Barriers and Beachwater Quality

Cost Category	Cost
Construction:	
Equipment and Supplies	$ 481,000.00
Technical Services:	
Preliminary Studies, Supervision Evaluation and Reporting	7,180.81
Operation and Maintenance:	
Material, Supplies and Direct Labor	60,730.00 18,100.00
Contingencies and Miscellaneous	181.86
Evaluation and Report	6,612.56
Total Cost	$ 141,070.21

Table [illegible]

Section V. Hypochlorination of Streams and Overflow

Cost Category	Cost
Construction:	
Contract, Equipment and Supplies	$ [illegible]
Technical Services:	
Preliminary Studies,	[illegible]
Supervision, Evaluation and Reporting	[illegible]
Operation and Maintenance:	
Materials, Supplies and Direct Labor	[illegible]
Administration, Legal and Fiscal	[illegible]
Contingencies and Miscellaneous	[illegible]
Evaluation and Report	[illegible]
Total Cost	$ [illegible]

Table I-X

Section VI Bacterial Growth

Cost Category	Cost
Construction:	
Contract Equipment and Supplies	
Technical Services:	
Preliminary Studies Supervision Evaluation and Reporting	$ 6,143.47
Operation and Maintenance:	
Material, Supplies and Direct Labor	4,195.14
Contingencies and Miscellaneous	100.00
Evaluation and Report	800.00
Total Cost	$ 11,139.41

Table [illegible]

Section III Overflow Reduction by Sewer Flushing

Cost Category	Cost
Construction	
Equipment and Supplies	
Technical Services	
Preliminary Studies, Supervision	$ [illegible]
Evaluation and Reporting	[illegible]
Operation and Maintenance	
Material, Supplies and Direct Labor	
Contingencies and Miscellaneous	[illegible]
Evaluation and Report	[illegible]
Total Cost	$ [illegible]

Table I-8

Section VIII Sewer Overflow Reduction Using Polymers

Cost Category	Cost
Construction:	
Equipment and Supplies	
Technical Services:	
Preliminary Studies, Supervision	$ [illegible]
Evaluation and Reporting	[illegible]
Operation and Maintenance:	
Material, Supplies and Direct Labor	[illegible]
Contingencies and Miscellaneous	[illegible]
Evaluation and Report	[illegible]
Total Cost	$ [illegible]

Table [illegible]

Section [illegible] Screening Devices

Cost Category	Cost
Construction:	
Equipment and Supplies	$ [illegible]
Technical Services:	
Preliminary Studies, Supervision, Evaluation and Reporting	
Operation and Maintenance:	
Material, Supplies and Direct Labor	[illegible]
Contingencies and Miscellaneous	[illegible]
Evaluation and Report	[illegible]
Total Cost	$ [illegible]

Table [illegible]

Section [illegible]. Debris and Solids Collection

Cost Category	Cost
Construction:	
Equipment and Supplies	$ [illegible]
Technical Services:	
Preliminary Studies, Supervision, Evaluation and Reporting	
Operation and Maintenance:	
Material, Supplies and Direct Labor	[illegible] [illegible]
Contingencies and Miscellaneous	[illegible]
Evaluation and Report	[illegible]
Total Cost	$ [illegible]

PB258192

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY
CINCINNATI, OHIO 45268

SUBJECT: Report for Inclusion in NTIS | DATE: September [illegible], 1976

FROM: Gilbert M. Gigliotti
Director, Technical Information Staff

TO: Input Branch
National Technical Information Service
U.S. Department of Commerce
Springfield, Virginia 22161

Enclosed is one copy of "Demonstration of Interim Techniques for Reclamation of Polluted Beachwater," by James F. Weber (EPA-600/2-76-[illegible]) for inclusion in the National Technical Information Service. All required information and forms are included.

~~We are submitting our only file copy of this report and would appreciate receiving a complimentary copy after your printing.~~

If you have any questions or need any other information, please call Mrs. Thomasine Bayless (513/684-[illegible]). Thank you for your cooperation.

Enclosure